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Proceedings of the American Society of Civil Engineers

CONTENTS

December, 1959

Papers

(Over)

Page

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

TYPE FEATURES OF THE OHIO SHORELINE OF LAKE ERIE

Howard J. Pincus[1]

SUMMARY

Using aerial photographs for illustration, features of the Ohio section of Lake Erie's shoreline are examined in the following major categories: I—Elongated, sandy bodies of low relief; II—Lowland areas; III—Mouths of streams; IV—Bluffs; V—Artificial shorelines. Associated beach types are tabulated. The orientation of incident wave energy is correlated with some types of shore features.

INTRODUCTION

The purpose of this paper is to present a systematic treatment of the engineering geology of the Ohio shoreline of Lake Erie through the medium of an outline classification of shore features.

The classification is intended to be analytical; complete areal coverage is not attempted here.

Many of the entries in the classification are illustrated by means of aerial photographs (Fig. 1, 5-17).

ACKNOWLEDGMENTS

The Ohio Division of Shore Erosion has borne almost all of the expense of sponsoring the author's studies of shoreline processes along Lake Erie since the inception in 1950 of this research program. Aid and cooperation have

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been freely given by many individuals and local, state, and federal agencies; contributions of the Ohio State University and its Development Fund are especially noteworthy.

For all of this assistance, the author is deeply grateful.

The Physical Setting

The general geology and topography of the Lake Erie basin are competently treated in detail in several widely known references (Leverett, 1902; Leverett and Taylor, 1915; Carman, 1946; Hough, 1948). In order to set the stage for the discussion which follows, a brief exposition of the physical setting follows; references cited here will provide gateways for the reader desiring more detailed information.

Bedrock

All of the bedrock exposed along the shoreline of Lake Erie and underlying its basin is sedimentary. Within the Ohio section, the Sandusky area marks the division between relatively resistant Silurian carbonate rocks to the west, and relatively non-resistant Devonian shale and shaly sandstones to the east (Fig. 2); a belt of resistant Devonian limestone underlies the Sandusky area itself. Rock is exposed along less than one-sixth of the Ohio section of the shoreline.

The resistant rocks are distributed in an arcuate pattern which is convex northward, the rocks dipping outward from the arc; the outer belts extend northward from the vicinity of Sandusky, their presence being marked by the rugged shoreline areas of Marblehead and Catawba Peninsulas, and the broadly curved belt of islands and shoals to the north and northwest. The belts of non-resistant rocks extend from the south, where they strike parallel to the

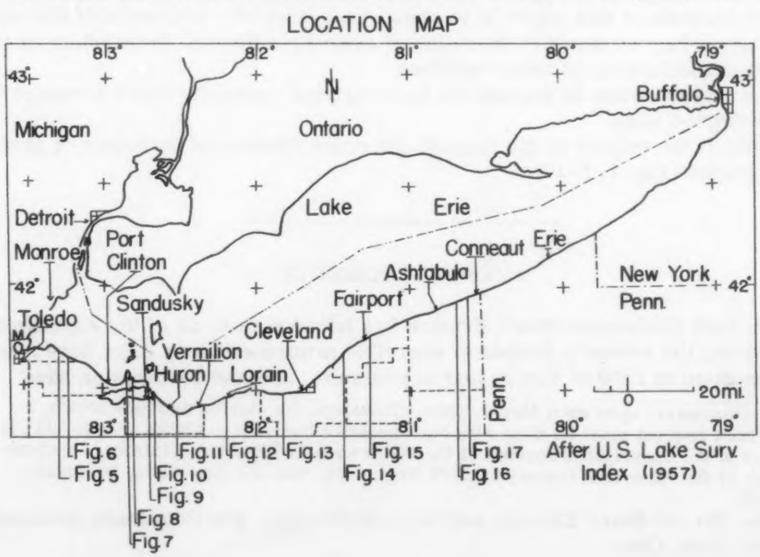


Fig. 1. Map showing locations of areas depicted in Figs. 5-17.

belts of resistant rocks, but in the vicinity of Sandusky the non-resistant belts swing almost due eastward, along the same path as the shoreline to the east.

To the south, only a short distance from the shoreline, the non-resistant rocks are bounded by relatively resistant Mississippian sandstones and shaly sandstones, which, with the underlying Devonian rocks, dip gently southward. The northward facing escarpment of Mississippian rocks is also the northern boundary of the Appalachian Plateau (Fig. 3).

The orientation of the lake basin has almost certainly been determined by the areal distribution of non-resistant and resistant rocks (Carman, 1946, and Fig. 2), the former having provided a relatively easy pathway for the southwestward advance of glacial ice, which scoured the bedrock en route. Carman (1946) attributes the differences in depth of the Western, Central, and Eastern Basins (Fig. 3) to differences in the bedrock's resistance to glacial scour. The shallowness of the Western Basin, which has a mean depth of 24.2 feet, is due presumably to the relatively high resistance of the underlying carbonate rocks. The Central and Eastern Basins owe their greater mean depths (60.7 and 79.9 feet, respectively), to the lower resistance to scour of shales and shaly sandstones underlying part of their areas.

Fig. 2. Map of resistant and non-resistant rocks in the Lake Erie region.

Surficial Deposits

Glacial and glacial lake deposits comprise virtually all of the surficial materials exposed along the shoreline. The glacial deposits consist of tough boulder clays, or till, composed of particles of a wide range of sizes and containing fragments of several types of rock. Bottom deposits of lakes ancestral to Lake Erie consist of clay, silt and fine sand. The flat plain sloping toward the Western Basin (Fig. 4) is the surface expression of such materials, exposed with the northeastward retreat of ancestral lake waters. Some of the shoreline deposits of ancient lakes form sandy ridges lying to the south of and in general parallel to the present shoreline.

Bluffs composed completely of surficial materials stand up to approximately 70 feet high. In some areas the bluffs consist exclusively of till or of lacustrine (lake) deposits, while in others both types of deposits are represented, at times in very varied vertical sequences.

The engineering properties of the soils in this area are very clearly related to the composition of the underlying bedrock, and they reflect the lacustrine origin of the sediments which have been their principal source material (Fig. 3).

Nearshore Bottom Materials

Much of the bottom of Lake Erie consists of a mud which is a semi-fluid mixture of silt and clay. In the nearshore zone lying lakeward of an area of sandy beaches, one can usually find some sand, although it is usually only a veneer. Similarly, where bedrock is exposed in the bluff or at lake level, it is usually also found to extend at least a short distance lakeward.

Off the southern shore of the Western Basin, where the sand disappears lakeward, there is a band of lacustrine clay parallel to the shoreline and very much like the clay which appears in the land area immediately to the south.

Relatively large deposits of sand lie along or near the shoreline off Fairport, Sandusky (outer Sandusky Bay), and Toledo (outer Maumee Bay).

Shale appears in the nearshore zone from just east of Fairport to the Pennsylvania border and beyond; there are some sections in this stretch of shoreline in which the shale does not appear in the bluff or at water level.

Sources of Beach-Building Materials

Materials exposed along the shoreline are, in general, very poor sources of beach-building sediments.

The limestones and dolomites which are exposed at a few places around the Western Basin furnish fragments which pile up near their sources as boulder, cobble, and pebble beaches; such fragments appear farther from the source as a much diluted, relatively sparse fraction of sandy beaches. Some of the thin-bedded limestones and dolomites yield carbonate shingle.

The shales, which are exposed far more abundantly than are their associated siltstones and sandstones, furnish platy and flaggy materials which accumulate in dark shingle beaches near their sources; they supply smaller flat fragments, which sometimes give beach sediments a dirty appearance, farther from the source. Fragments of shale are apparently destroyed fairly quickly by weathering and wear.

The surficial materials appearing in the bluffs contain relatively small amounts, say 10-20%, of beach-building materials, although toward the Ohio-

Pennsylvania border, bluff sections consist on the average of about 30% of beach-building materials.

From the tills, fragments of limestone, dolomite and shale are introduced into the littoral zone, but such materials are usually quickly diluted in the more abundant finer fractions of the littoral stream.

The commercial sand and gravel deposits lying in deeper waters, viz., off Lorain and Vermilion and off Fairport, do not appear to be sources of sediment in the modern littoral stream. Those deposits lying closer to the shoreline, viz., in outer Sandusky Bay and in outer Maumee Bay, seem to be depositories for rather than sources of littoral drift.

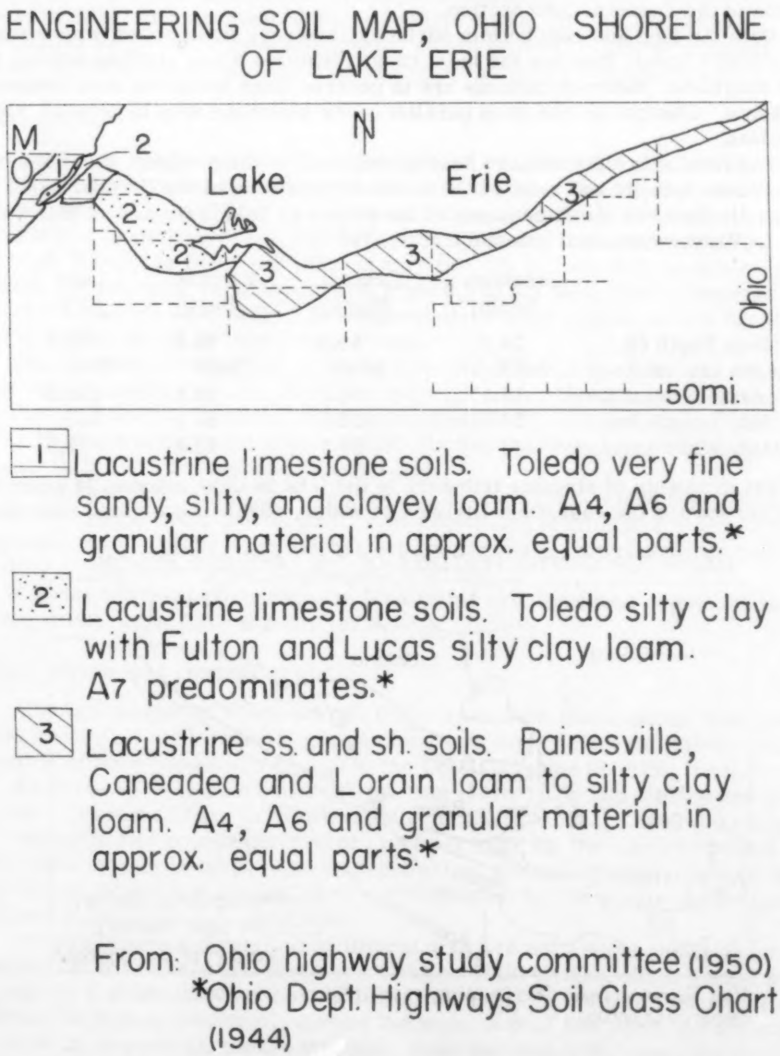


Fig. 3. Engineering soil map, Ohio shoreline of Lake Erie.

Streams entering Lake Erie in Ohio waters contribute little sand to Lake Erie (Ohio Division of Water, 1953). Even those streams traversing areas underlain by sand and sandstones do not transport much sand to the lake. Their drowned mouths, especially where developed as harbors, act as settling basins for all but the very finest sediments.

Topography

The Ohio land area adjoining the lake is part of the Eastern Lake Section (Fig. 4) of the Central Lowland Province, which in turn, is part of the Interior Plains (Fenneman, 1946). The Till Plains and Appalachian Plateau lie to the south of the Eastern Lake Section.

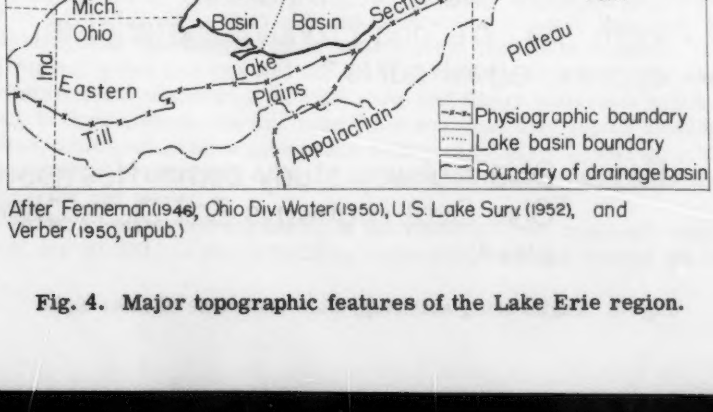
Both the land and lake bottom surfaces have very little relief; where shoreline bluffs occur, they are the most rugged features along profiles normal to the shoreline. Bedrock bottoms are in general more irregular than sediment bottoms. Changes in elevation parallel to the shoreline are, in general, very gradual.

The Ohio shoreline of Lake Erie includes all of the southern shoreline of the Western Basin and most of the southern shoreline of the Central Basin (Fig. 4). Some of the dimensions of the basins of Lake Erie are as follows (J. L. Verber, personal communication, 1950):

	Western Basin	Central Basin	Eastern Basin	Lake Erie
Mean Depth (ft.)	24.2	60.7	79.9	61.0
Area (sq. mi.)	1495	6410	2035	9940
Area (% total lake)	15.1	64.5	20.4	100.0
Max. Length (mi.)	50	132.5	85	241
Max. Width (mi.)	40	57.5	42.5	57.5

The gradients of streams tributary to the lake in Ohio become, in general, steeper toward the east (Ohio Division of Water, 1953). Associated with this

MAJOR TOPOGRAPHIC FEATURES OF THE LAKE ERIE REGION



After Fenneman(1946), Ohio Div. Water (1950), U.S. Lake Surv. (1952), and Verber (1950, unpub.)

Fig. 4. Major topographic features of the Lake Erie region.

contrast in gradients, and also with changes in soils and bedrock (Figs. 2 and 3), the average water yield (c.f.s./sq. mi.) of the streams' drainage basins increases from west to east.

Buried courses of ancient streams, like those of the ancestral Rocky and Cuyahoga Rivers, are of considerable significance in coastal engineering work because of the associated lows in the bedrock surface and the presence within the buried valleys of river and drowned embayment deposits, commonly composed of silt, sand, and clay (Ohio Division of Water, 1953A).

Within the belt of limestones and dolomites extending northward from north of Sandusky through the island area, there is little or no established surface drainage. Probably much of the drainage is taken care of by subterranean courses in solution openings in the carbonate rocks; further, the small catchment areas in the islands probably do not provide sufficient amounts of water for establishing surface drainage of even moderate proportions.

Lake Levels

Low water datum for Lake Erie, as defined by the U. S. Lake Survey, is 570.5 feet above mean tide at New York, 1935 datum. The 98 year mean lake level (1860-1957) is 572.36 above datum. The maximum difference between the highest monthly average (574.70 in May 1952) and the lowest monthly average (569.43 in February 1936) is 5.27 feet (U. S. Lake Survey mimeo reports); the seasonal variation ranges from 0.5 to 2.8 feet, with an average of about 1.6 feet (Saville, 1953). The highest and lowest monthly levels usually occur in June and February, respectively.

Wind-induced fluctuations in water level can be quite spectacular. During a severe storm in January, 1942, the recorded water level at Buffalo exceeded that at Toledo by a maximum of approximately 13-1/2 feet, only a little over 12 hours after water levels at both stations had been approximately equal (U. S. Lake Survey data, unpublished).

Personnel of the Ohio Division of Shore Erosion have logged longitudinal seiches, clearly identified as such, with amplitudes of from 1/2 to 2-1/2 feet, approximately. Transverse seiches apparently cause fluctuations approximately 1/10th those of the longitudinal seiches.

Factors affecting fluctuations in lake levels are discussed elsewhere in some detail (U. S. Lake Survey, 1952).

Wind, Waves, and Littoral Drift

Although southwest winds prevail in the Lake Erie basin, winds from the east, northwest and northeast can combine with large fetches to produce waves capable of delivering much energy to the Ohio shoreline (Saville, 1953).

Wave energies calculated from wind data (1948-1950) and fetches are given below. These data have been extracted from Saville (1953). W is the calculated energy in foot pounds per foot of crest per year for the ice-free period. The wave system is taken to be an hypothetical uniform system of waves of significant height and period. Data are calculated for deep water conditions, without corrections for refraction.

The predominant directions of littoral drift are not always easily ascertained, but it is likely that there is a division of directions within a zone from Lorain to a short distance eastward (Pincus, 1954). West of this zone the direction of drift is dominantly toward the west; east of this zone, the direction of drift is, in general, toward the east. Near and within the zone, the pattern of drift is usually quite erratic.

Monroe, Michigan		Cleveland, Ohio		Erie, Pennsylvania		Buffalo, New York	
Dir.	$W(10)^9$	Dir.	$W(10)^9$	Dir.	$W(10)^9$	Dir.	$W(10)^9$
NE	.192	W	.567	WSW	.440	SW	.408
ENE	.195	WNW	.804	W	1.227	WSW	2.827
E	.570	NW	.529	WNW	.478	W	2.036
ESE	.197	NNW	.769	NW	.598		
SE	.068	N	.327	NNW	.314		
SSE	.009	NNE	.543	N	.130		
		NE	.345	NNE	.363		
		ENE	.002	NE	.368		

Local reversals of drift and local neutral points occur within the areas of predominant eastward and westward drift. Such factors as changes in configuration of the shoreline or the presence of artificial structure favor incident energy from directions with components opposite to the regionally dominant direction.

Tilting of the Lake Basin

There is abundant evidence, such as drowned stream mouths, for recent submergence of the shores of Lake Erie; the submergence results from uplift of the lake's outlet with respect to its shores.

Studies of horizontal and tilted strand lines have led to a postulated pattern of hinge lines, which, in turn, has been taken as evidence for isostatic rebound following discrete retreats of continental ice sheets (Flint, 1947).

Differential water level data collected over a half-century indicate a direction of rotation of the land area which is consistent with the direction of rotation required by the glacial rebound hypothesis, however, instrument levelling data indicate sinking of the entire area with respect to mean sea level (Moore, 1948).

Whether the land is moving up or down with respect to mean sea level is of considerably less consequence here than is rotation which affects the relative elevations of the lake's shores and its outlet. The second type of movement has obvious geologic and limnologic consequences, and, in addition might lessen the long-term effectiveness of some structures designed to protect the shoreline. A major structure in a locality which is sinking at a rate of 1/2 ft./century with respect to the lake's outlet might be demonstrably less effective, although not necessarily ineffective within its planned lifetime, just because of submergence which had not been taken into account when the structure was designed.

Ice

Lake ice, pushed shoreward, may steepen lakeward slopes of unconsolidated shore features. With the onset of wave action and higher lake levels in the spring, this steepening disappears (Verber and Hartley, 1954, unpublished).

Such ice may pry off exposed bedrock; unlike the changes described above, the effects on bedrock are irreversible.

Ice moving in any direction may damage shore structures in many ways.

Where large areas of lake ice do not provide a buffer between open water and the bluffs, the ice coatings on bluff faces apparently provide protection against wave attack.

A Classification of Shore Features

Attempts to discuss shorelines in genetic terms and from the point of view of engineering problems quite naturally lead to some classification of shore features. Classifications of several types have appeared in the literature of geology and engineering; probably the best known are those of Gulliver (1899) and Johnson (1919 and 1925), and Shepard (1937), but the subject has also been treated by Gilbert (1885), Stewart (1945), McCurdy (1947), Powers (1954), Price (1954), Guilcher (1954), and others. The Beach Erosion Board's "Type Classifications," File S-02-076, in the mimeographed paper, "General Studies of Shore Processes in the Great Lakes" is concerned specifically with lake features.

The classification used in this paper shares more common elements with the systems used by Powers (1954) and the Beach Erosion Board (S-02-076) than with the others, but all of the references cited above have had some influence on the author's thinking.

For some of the terminology of Type I below, the author has drawn upon Shepard's (1952) discussion, in which the emphasis is placed upon the geometry of depositional coastal features, rather than on their inferred origin.

Figs. 5-17 are aerial photographs showing examples of most of the features appearing in the classification. For ease in locating the features they depict, the figures are numbered from west to east (Fig. 1), but they make their appearance in the classification in a different sequence. Technical data on these photographs appear in the Appendix.

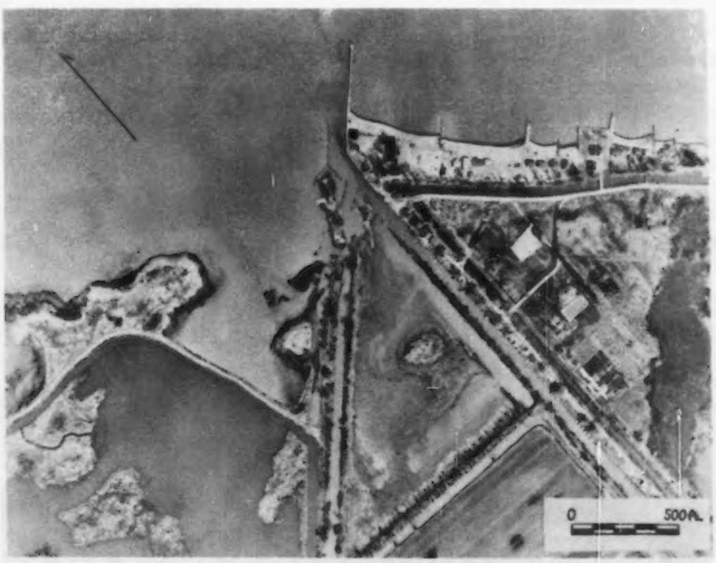


Fig. 5. Mouth of Cooley Creek.

Outline Classification

Types of Shore Features, Ohio Section of Lake Erie

Dominant Feature	Illustration	Remarks
I. Elongated, dominantly sandy bodies of low relief, lying essentially parallel to the trend of the lake bottom contours. Generally some water or marshy areas lie between these bodies and the mainland.		These features lie lakeward of lowlands, drowned areas, etc. (See Type II). Offshore topography slopes gently lakeward, with a sandy cover which may wedge out or grade into finer sediments downslope. These features are depositional, but some of their surfaces may be erosional.
a) Barrier beach Single ridge parallel to mainland shore, and separated from it by lagoon.	Fig. 6	The feature shown is moving s.w. toward the lagoon. In some other localities, a dike has been constructed along the side facing the lagoon, preventing landward migration, and resulting in the development of a steep slope and occasionally a scarplet on the lakeward side.

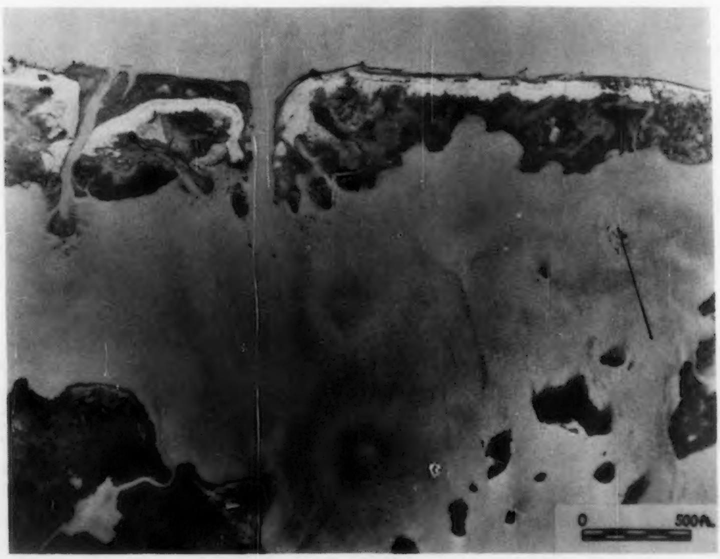


Fig. 6. Magee Marsh area, just east of Crane Creek State Park.

Dominant Feature	Illustration	Remarks
b) Barrier island Multiple ridges, parallel to mainland shore; zones of vegetation and swampy areas extending lagoonward.	Fig. 8	The feature in the upper left is connected to the mainland by a causeway located to the northwest, out of the picture. Gravel Bar, on the right side of the picture, has been much modified by man; formerly it was probably a combination of I a) and I b).
c) Barrier spit Same as above, but anchored to mainland. (Artificially anchored barriers are not included here.)	Fig. 10	Only the northwestern tip of the compound spit is shown; portions of the spit not shown here display some of the features of I a) and I b).
d) Bay spit Anchored to mainland, and growing across mouth of bay (or drowned river or bay mouth), not parallel to the mainland shoreline.	Fig. 9	Littoral currents promote southeastward growth of this feature, but it is not likely that this will develop into a bay barrier (I e)) because of vigorous flushing in the channel to the southeast.
e) Bay barrier Bay spit has grown across mouth of bay (or drowned river or bay mouth).	Fig. 11	Feature at left breached at time of photography. Occurs intermittently at drowned mouths of small streams without structures. Feature would occur at drowned mouths of most major streams emptying into the lake, were it not for the works of man.
II. Lowland areas of extremely low relief, consisting of bottom sediments, chiefly silt and clay, exposed when the waters of an ancestral lake retreated.		This is the type of area often found landward of I a), b) and c), and separated from them by lagoons and marshy areas. The illustrations given below face the water with no Type I features in front of them.
a) Lake shore	Fig. 5	Note the contrast between the unprotected, undrained area to the left, and the protected and drained area to the right.

Dominant Feature	Illustration	Remarks
b) Bay shore		In the upper part of Sandusky Bay, along the south shore, beds of marl exposed at storm water level permit very rapid erosion, even for storms of small fetch (Bowman, 1953). Very little beach building material is available here; there is no way of tapping the lake's littoral stream.
III. Mouths of streams		Outflow, deposition, and structures impede or intercept littoral drift; sandy littoral stream probably gains very little from stream sources.
a) Emptying into lake through drowned mouth		
1) Flanked by lowlands	Fig. 5	
2) Flanked by bluffs	Fig. 13	Shale bluffs and structure protect mouth of stream.

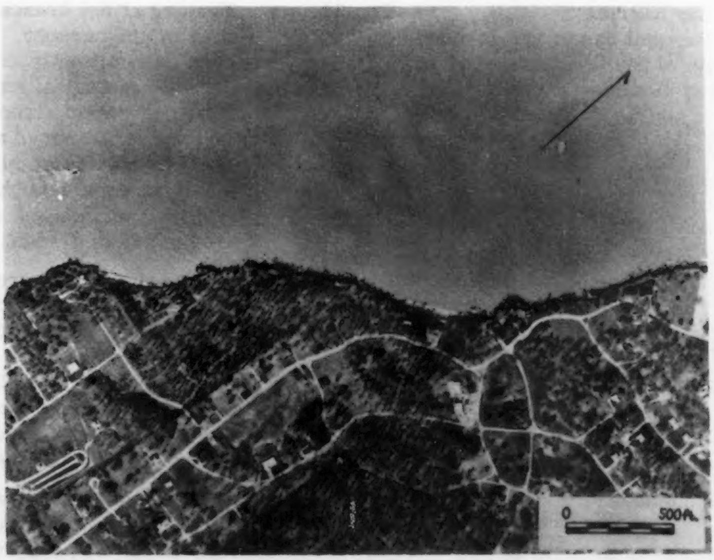


Fig. 7. Northwestward facing rocky bluffs, Catawba Peninsula.

Dominant Feature	Illustration	Remarks
	Fig. 16	Structure (wall) left of stream mouth and bedrock "high" near lake level impede recession of the shoreline. Stream mouth probably kept flushed because of confined flow. Till bluffs provide no direct protection for mouth of stream.
b) Emptying into lake through drowned bay		Sandusky River flows into Sandusky Bay, which opens into the lake proper just beyond the pass between Sand Point (Fig. 9) and Cedar Point (Fig. 10); see Fig. 1.
c) Emptying into lake through channel cut in sediments deposited in former local embayment by ancestral lake waters at higher level.	Fig. 14	West branch kept open by structure on west side of mouth and by dredging; east branch has been barred across.
d) Emptying into lake through mouth which is well-protected by structures.		Mouths of all major streams are well-protected. An example of a large protective structure appears in the right part of Fig. 12; this is part of the west breakwater at the mouth of the Black River, Lorain.
IV. Bluffs		Wave attack and ice push and heaving at the toe of bluffs can cause undermining.
a) Rock		Pattern of natural quarrying controlled by jointing, relative resistance and thickness of bedding, etc.
1) Dolomite and limestone	Fig. 7	Shoreline is sinuous, with pocket beaches of coarse, rounded fragments derived from local sources. Rocks often break off in blocky slabs. Some beds and their fragments become pitted by solution.

Dominant Feature	Illustration	Remarks
2) Shale, with some siltstone and sandstone.	Fig. 13	Shoreline is roughly scalloped in some areas where shale is exposed. Irregularities in the plan view of shale bluffs often reflect the presence of minor faults and folds. Shingle beaches are made up of fragments derived from thin-bedded rocks exposed in the bluffs. Shale fragments are broken down fairly rapidly by chemical weathering and mechanical wear.
b) Surficial materials		Beach may lie at toe of bluff; this is more likely in areas in which the bluffs can furnish more than traces of beach-building materials. Beach protects toe of bluff.
1) Boulder clay (till)	Fig. 12	Fairly well protected till bluff; upper (weathered) part of till is yellow-brown; lower part is blue-gray.

Fig. 8. East Harbor inlet, with Gravel Bar development at the right and East Harbor State Park at the upper left.

Dominant Feature	Illustration	Remarks
	Fig. 15	Bluffs consist chiefly of till; probably also some lake clay in the bluff. Bedrock at least 20 ft. below water level. Note slabs of roadway which have settled downward.
	Fig. 14	The C.E.I. plant at the left rests upon a till section, with patchy capping of old lake sediments; the till bluff in this area is well protected. To the right of the road left of center are lower lying lake deposits.
		Slope wash over these impermeable materials accounts for much erosion; vegetation on graded surfaces is often an effective counter measure.
2) Lacustrine (lake) sediments (silt, clay and sand)	Fig. 11	Laminations are often contorted in confusing patterns; lowest few feet are till.
		When saturated and subjected to pressure from above, silts commonly flow outward and downward in lobate pattern. Face of bluff consisting of alternating silts and clays often shows a miniature step-and-riser profile.
3) Combinations of 1) and 2)	Fig. 16	From bottom to top, bluffs in this vicinity show till, laminated clay, fine sand, and till. Bluffs fail by flowing and dropping off in blocks; note steep upper portion and scalloping.
	Fig. 17	From bottom to top, bluffs show till, lacustrine clay, till, lacustrine clay, sand, and loam. Bluff fails by sloughing.

Dominant Feature	Illustration	Remarks
		While bluffs showing only till often fail along vertical surfaces, those bluffs in which till lies upon at least several feet of lake deposits, particularly silts, often fail by slumping of the till along curved surfaces and flowing of the silt beneath.
c) Combinations of a) and b), with surficial materials lying on top of bedrock.		Presence of bedrock, even if only at water level, impedes retreat of bluff.
1) Top of bedrock approximately at lake level.		Salient at mouth of stream in Fig. 16 is due in part to bedrock "high." Salient in right portion of Fig. 17 is very much like others beneath which bedrock "highs" have been found, although this salient has not yet been probed.
2) Top of bedrock well above lake level.		Rock bluffs shown in Fig. 7 and Fig. 13 are actually capped with surficial materials, but so much of the sections are composed of bedrock that the surficial

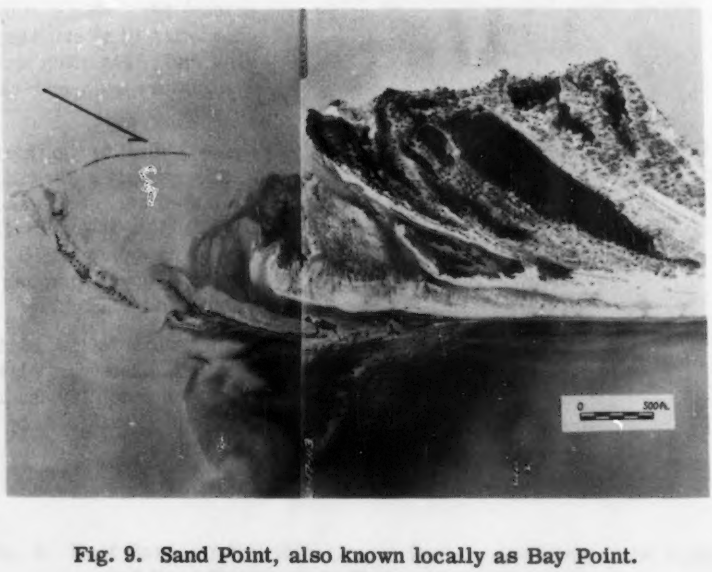


Fig. 9. Sand Point, also known locally as Bay Point.

Dominant Feature	Illustration	Remarks
		materials exercise no demonstrable control over development of the bluffs.
V. Artificial shorelines		Works of man completely dominate the shoreline, as in the shoreline of Cleveland.

Additional Comments on Shore Features

The shore features considered above have, for the most part, arrived at a stage of development beyond which they should not change very much in size or form within the next few generations, although the location of the shoreline may be displaced considerably. However, works of man can exercise very large influence locally, and it is likely that for better or worse, the major changes will be man's doing.

In connection with the overall map configuration of shorelines, Shepard (1937) asserts that waves can straighten shorelines very quickly, and that uniformity of materials favors linearity. Linearity also develops where alternating bands of resistant and non-resistant materials lie parallel to the shoreline. He states also that wave action causes irregular shorelines where materials of different relative resistance are exposed. All of these assertions are well supported by the author's observations along Lake Erie's shoreline. In fact, these relations are quite useful for interpreting aerial photographs and maps by those who have not had the opportunity to make field inspections.

Obviously the analytical methods of soil mechanics can throw much light upon the development of Type IV features. Chieruzzi and Baker (1958) have initiated such work along the Lake Erie shoreline, and have presented suggested outlines for further study.

A Rough Classification of Types of Beaches, Ohio Section of Lake Erie

As an auxiliary to the preceding outline classification of shore features, the following rough classification of types of beaches is presented. While the classification below seems at first glance to be based upon particle size and shape alone, a second glance will show that the occurrences of each of these types bear systematic relationships to some types of shore features discussed earlier.

Beach slopes are not included here partly because many Lake Erie beaches consist of sequences of thin layers of significantly different particle sizes, each layer's upper surface slope affecting the slope of the overlying veneer. Further, for the purposes of this paper, the addition of available beach slope data, even for relatively homogeneous beaches, would not refine the following outline materially.

Composition	Illustration	Types of Associated Shore Features	Remarks
a) Sand			
1) Elongated bodies		I	Consist chiefly of fine sand, well sorted. Some include some medium sand, and even gravel.
	Fig. 6	I a)	
	Fig. 8	I b)	
	Fig. 9	I d)	
	Fig. 10	I c)	Jetty at northwestern end was constructed after the main part of the spit had been formed.
	Fig. 11	I e)	Left part of photo.
2) Beaches at toes of bluffs	Fig. 17	IV b)	These beaches (center of photo) are more likely to occur where erosion of bluff materials passes

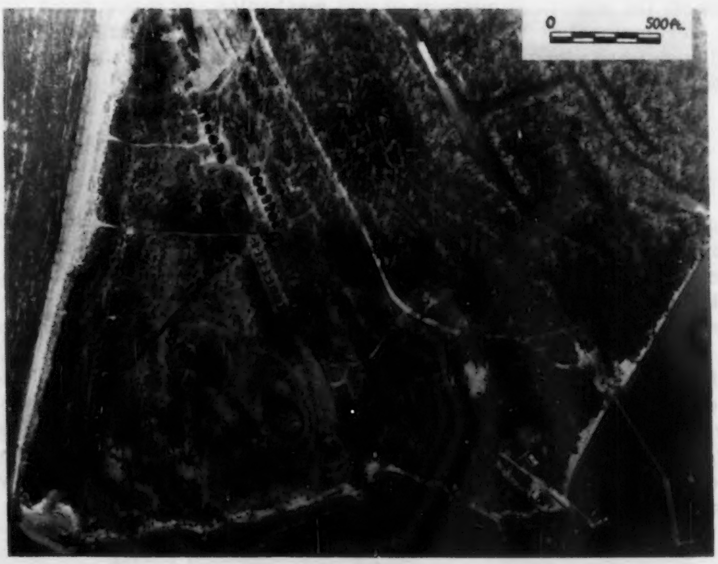


Fig. 10. Northwestern tip of Cedar Point.

Composition	Illustration	Types of Associated Shore Features	Remarks
			beach-building sediments to littoral stream.
3) Salients	Fig. 17	IV c) 1)	Right side of photo. Elevation of bedrock not determined.
	Fig. 16	IV c) 1)	May be partly due to seawall; at least partly due to bedrock "high".
4) At stream mouths	Fig. 14	III c)	Structure at mouth of stream constructed to help keep passage navigable.
5) Artificially trapped or protected by seawall	Fig. 5	II a)	
	Fig. 12	III d)	Sediment swept into pocket east of west breakwater; note arcuate bar inside

Fig. 11. Old Woman's Creek and vicinity of Ceylon Junction.

Composition	Illustration	Types of Associated Shore Features	Remarks
			harbor breakwater, east side. Sediment also accumulated in groin field at Lakeview Park, to the west.
	Fig. 15	IV b) 1)	To left of center.
	Fig. 16	III a) 2), IV c) 1)	Protected by seawall to left of stream mouth.
	Fig. 17	IV c) 1)	To left of center.
a) Pebble, Cobble, Boulder			Generally pocket beaches, although some have been trapped by structures.
1) Dolomite and limestone exposed in bluffs.	Fig. 7	IV a) 1)	
2) From rock fragments in till, exposed in bluffs.		IV b) 1)	Just west of Vermilion (Fig. 1) there was a pebble

Fig. 12. Lakeview Park, Lorain, and west breakwater of Lorain harbor.

Composition	Illustration	Types of Associated Shore Features	Remarks
			and cobble spit in 1958 which very likely was supplied by local till sources.
c) Shingle			Shale fragments not durable.
1) Shale, exposed in bluffs; also thin-bedded siltstone and sandstone.		IV a) 2)	West of Huron (Fig. 1), and vicinity of Avon Point (app. Long. 82° W.)
2) From rock fragments in till, exposed in bluffs.		IV b) 1)	

Orientation of Incident Energy at the Shoreline

An understanding of the development of some of the shore features discussed earlier requires some type of analysis of the wave energy incident upon the shoreline.

The following outline, which is drawn up according to directional aspects of incident wave energy, might prove to be helpful in this regard. The direction of the incident wave energy has been inferred from such evidence as

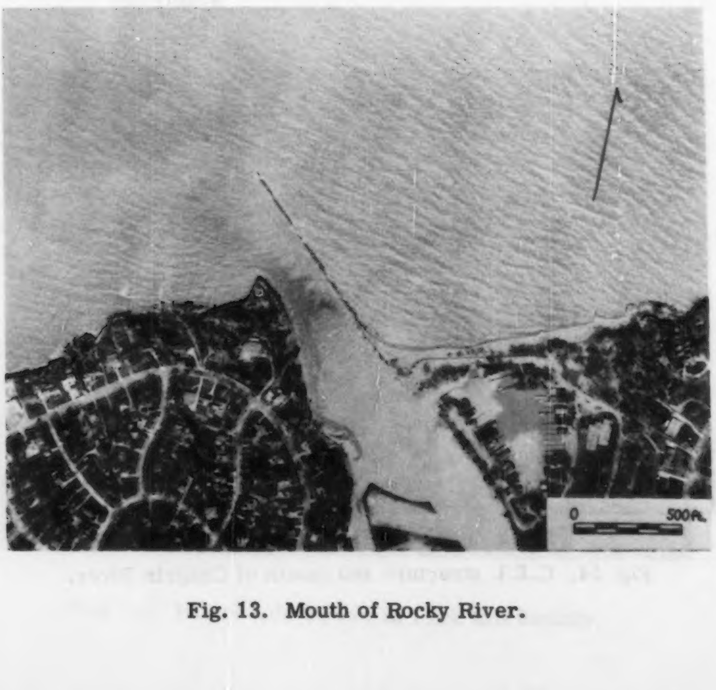


Fig. 13. Mouth of Rocky River.

accretion patterns, wind and fetch, variations in sediment parameters, etc. (Pincus, 1954).

Orientation	Illustration	Remarks
a) Incident energy parallel to shoreline		
1) Alternating		
i - Balanced		Assumed to be equivalent, in effect to b) 1) below, until more detailed data on incident wave energies are available.
ii- Unbalanced (transitional to 2) below	Fig. 9	Elongation due to unbalance
	Fig. 14	Asymmetrical accretion due to unbalance.
	Fig. 15	Asymmetrical accretion due to unbalance.
	Fig. 17	Asymmetrical accretion due to unbalance.
2) Direct pulsating (transitional from ii above)	Fig. 12	Inside breakwater, on right side of photo; sediment moved from west and southwest.

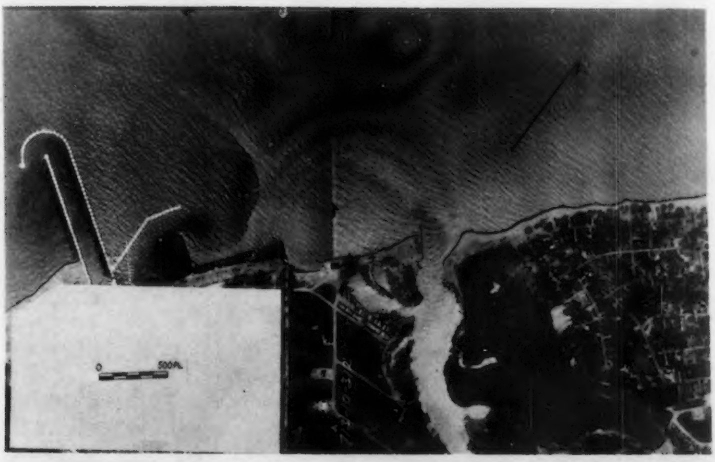


Fig. 14. C.E.I. structure and mouth of Chagrin River.

Orientation	Illustration	Remarks
b) Incident energy normal to shoreline		
1) Normal component plus balanced alternating (a) 1) i) yield normal resultant.		See a) 1) i remarks. Entrapment of some sediment by groins is possible.
	Fig. 12	In groin field at Lakeview Park, left half of photo; symmetrical accretion.
	Fig. 11	In front of bluff, right half of photo.
2) Normal component		Northwestward extension of feature in upper left corner of Fig. 8, viz., East Harbor State Park.

Further Comments on Incident Energy and Related Factors

The advance or retreat of a shoreline depends upon factors such as the orientation of incident energy, discussed above. Another factor entering the picture is the net gain or loss of sediment by a sedimentary body. The combination of these two factors can yield a variety of results, even if water level remains unchanged. Thus it is possible for the areas of transverse cross sections of elongated sand bodies to increase, but to have landward movement of the lake shoreline; this will occur if the normal component of incident

Fig. 15. Painesville Township Park and vicinity.

energy is sufficient to cause landward deflection in excess of lakeward growth. This has happened at times during the history of the areas shown in Fig. 6 and Fig. 8; the landward deflection is exaggerated in the inlet areas.

Further, it is possible for a feature's transverse cross-sectional areas to remain relatively unchanged, or even to have them decrease, but to have also lakeward movement of the shoreline facing the lake. This situation can arise with lakeward deflection of the littoral stream and erosion of the lagoon or bay side of the feature where large interior fetches are available. This has probably happened at least several times during the history of Sound Point (Fig. 9).

The situation can be further complicated by the works of man. For example, if the landward movement of a sand body is obstructed by a dike on the lagoon side, the shape and perhaps the areas of transverse cross-sections will change (often decrease), usually with a landward movement of the lake shoreline. This has happened in several localities bordering the Western Basin.

Concluding Remarks

The analysis presented in the preceding pages is clearly more descriptive than genetic. This emphasis arises from circumstance, not from choice. Although shoreline processes in many places have been studied for many years, truly rational analysis has not yet been achieved. From the fields of hydrodynamics and soil mechanics will come the necessary rational analysis; perhaps the empirical approach, as used in this paper, will be of some value until that time, and perhaps such empiricism will provide the necessary

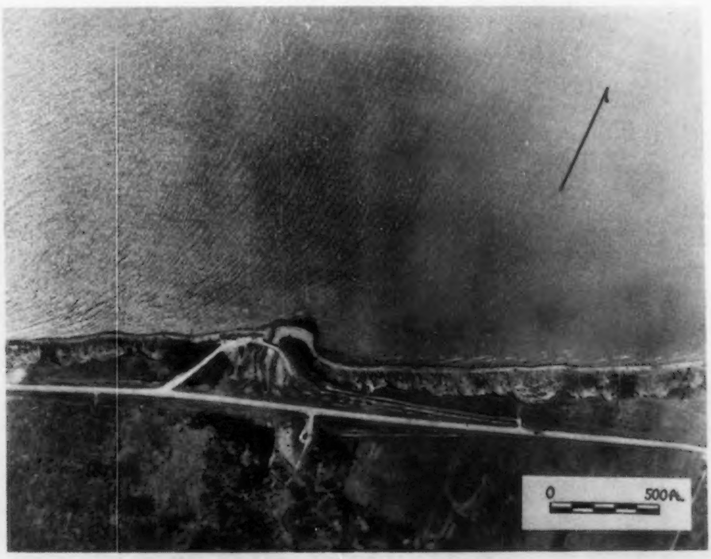


Fig. 16. Whitman Creek and vicinity.

observational data for testing conclusions built upon the deductive approach.

Many more observational data than are presented here have been collected. Some time during 1959 the Ohio Division of Shore Erosion will publish a series of seven maps giving the detailed description of the engineering geology of the entire Ohio shoreline of Lake Erie; subsurface sections, bottom deposit data, and other related information will be included. This set of maps is to be systematic in areal and encyclopedic terms; the paper before you is intended to be systematic in terms of synthesis.

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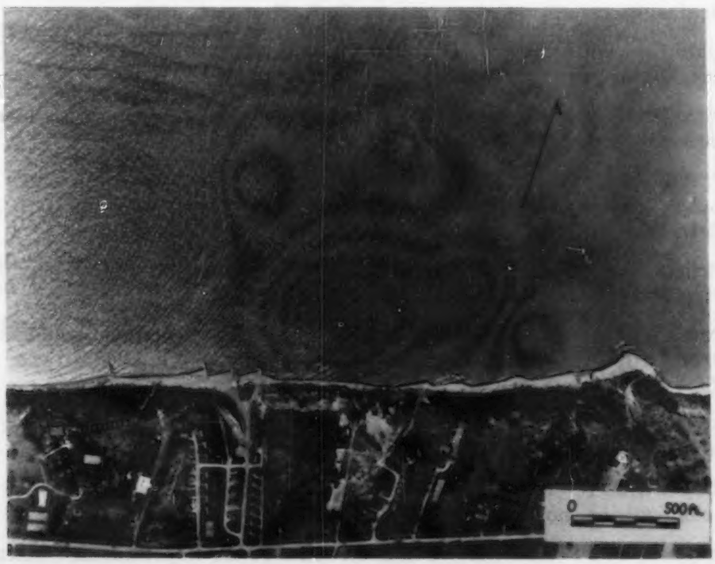

Fig. 17. Vicinity of Kingsville-Conneaut Township line.

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APPENDIX

Data on Aerial Photography

Aerial photography by Ohio Highway Department for Ohio Division of Shore Erosion.

Project 850, flown April 15, 1957.
8-1/4 in. lens at 3900 ft.
Water level at Toledo 572.2 ft. above m.s.l.

Figure No.	Aerial Photo Serial No.
5	850-33-314
6	850-30-275
7	850-24-177
8	850-21-147
9	850-17-111, 113

Project 713, flown May 8, 1956.
8-1/4 in. lens at 3900 ft.
Water level at Sandusky 573.3 ft. above m.s.l. (9 A.M.)
Wind S.E.—14 m.p.h.

Figure No.	Aerial Photo Serial No.
10	713-1-2
11	713-1-49
12	713-2-117
13	713-6-205
14	713-10-310, 312

Project 978, flown May 1, 1958.
8-1/4 in. lens at 3900 ft.
Water level at Fairport 571.6 ft. above m.s.l.
Wind W.S.W.—21 m.p.h. (12:30 P.M.) at Ashtabula.

Figure No.	Aerial Photo Serial No.
15	978-2-160
16	978-1-43
17	978-1-25

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Journal of the

WATERWAYS AND HARBORS DIVISION

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DEEP-DRAFT NAVIGATION ON THE GREAT LAKES[a]

Walter E. McDonald[1]

SYNOPSIS

The Great Lakes deep-draft navigation system comprises the harbors and the channels connecting Lake Erie with the Upper Great Lakes. The St. Lawrence Seaway, comprising construction of canals and deepening channels in the St. Lawrence River from Montreal to Lake Ontario and deepening the Welland Canal from Lake Ontario to Lake Erie, now forms an integral element of the overall deep-draft navigation system on the Great Lakes.

The Great Lakes Connecting Channels have been deepened and improved over the past 100 years primarily because of waterborne transportation economies in movement of iron ore, limestone, grain and coal serving the economic complex of the midwestern United States.

This paper describes the character and scope of studies undertaken by the Corps of Engineers, U. S. Army, for the present program for deepening the Connecting Channels and harbors to serve adequately the modern bulk cargo vessel fleet on the Great Lakes and also to determine navigation improvements required at Great Lakes harbors to accommodate vessel traffic through the St. Lawrence Seaway. These studies are the latest of a continuing effort in response to Congressional directives to the Corps of Engineers in determining the nature and extent of additional improvements warranted in the public interest to effect the maximum net benefits from waterborne commerce on the Great Lakes.

INTRODUCTION

The principal justification for participation by the Federal Government over the past 100 years in improving the deep-draft navigation system on the

Note: Discussion open until May 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2298 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 4, December, 1959.

a. Presented at the May 1959 ASCE Convention in Cleveland, Ohio.

1. Chf. Planning and Reports Branch, Eng. Div., U. S. A. Engr. Div., North Central Chicago, Ill.

Great Lakes has been the economies effected by waterborne commerce of iron ore from the Lake Superior district, limestone from the northern portions of Lakes Michigan and Huron, grain from Lake Superior ports, coal and petroleum from the Chicago and Lake Erie areas, all destined to serve the industrial complex and needs of our country. The extent of these economies has been largely in direct proportion to the use of larger and more efficient bulk cargo vessels which have been constructed over the years at a pace in keeping with vessel operating conditions encountered in the channels connecting the Great Lakes and in the harbors.

In the 1930's the Great Lakes Connecting Channels between Lake Erie and the Upper Lakes were deepened to provide a controlling depth of 25 feet in two-way and downbound channels (Lake Superior to Lake Michigan and Lake Erie) and 21 feet in the upbound channels. (Lake Erie to Upper Lakes). These controlling depths still prevail. The shipping and receiving harbors serving the major share of the waterborne traffic in iron ore, stone and grain were also deepened during the period 1935-1945 to depths commensurate with the downbound Connecting Channels.

The District Engineer, Corps of Engineers, at Detroit prepared in 1944 a review survey report in response to a request by Congress to determine whether at that time modification of the Connecting Channels was advisable in the interest of the needs of then existing and then anticipated navigation. The economic analysis developed by the District Engineer for the plan of deepening the Connecting Channels as considered in the 1944 review report noted that during the period between 1930 and 1944 the maximum bulk cargo vessel on the Great Lakes increased in length from about 550 to 640 feet overall. The largest vessels in operation in 1943 had a carrying capacity at maximum loadline limit of about 18,500 tons. Also, these vessels had maximum loadline draft of about 24 feet. Five vessels of about this size were placed in commission in 1942.

The safe vessel draft at low water datum that prevailed in 1944 for the downbound Connecting Channels was 22.3 feet. The District Engineer in his review report stated that only 30 vessels of the fleet of Great Lakes cargo carriers as then constituted (1944) had summer season drafts in excess of 22.3 feet. The conclusion was then reached that the fleet as constituted in 1944 would be reasonably representative of vessels of the fleet for the ensuing years, as determined by discussions with vessel operators, and would not benefit from further deepening of the Connecting Channels and harbors to a sufficient extent to justify economically the very large cost of the deepening work. The recommendation transmitted to Congress by the Chief of Engineers was that deepening of the Connecting Channels-Harbors system not be undertaken at that time.

In 1953 the Public Works Committees of the United States Senate and House of Representatives adopted similar resolutions authorizing the Corps of Engineers to review previous reports on the Great Lakes Connecting Channels, and associated deep-draft harbors, to determine the advisability of further deepening and improving the Connecting Channels, including consideration of providing depths of at least 27 feet.

Engineering and economic studies by the Detroit District led to a favorable recommendation for deepening the Connecting Channels from the existing controlling depth of 25 feet in downbound channels and 21 feet in upbound channels to a controlling depth of 27 feet in both downbound and upbound channels. These recommended improvements were authorized by Act of Congress

approved 21 March 1956. The project deepening is now under construction and with full 27-foot controlling depth in all channels scheduled for completion in 1962.

In May and June of 1956 the Public Works Committees of the United States Senate and House of Representatives adopted resolutions requesting the Corps of Engineers to review the previously completed report on the Connecting Channels to determine the advisability of further improvement of specific harbors on the Great Lakes in the interest of present and prospective deep-draft commerce and that due regard be given to the scheduled time of completion of the St. Lawrence Seaway, which was then under construction, and of deepening of the Connecting Channels above Lake Erie.

The District Engineers, Corps of Engineers, at St. Paul, Chicago, Detroit and Buffalo are now making, under the supervision of the Division Engineer in Chicago, interim studies on individual harbors. The comprehensive Great Lakes Harbors Study is well under way, with interim reports on several harbors, or portions thereof, already completed.

This paper will discuss the character and scope of studies which were required to reach sound conclusions as a basis for determining improvements justified for the Great Lakes Connecting Channels and of the similar, and more detailed, studies now under way to determine what improvements are justified at Great Lakes Harbors to accommodate the deep-draft vessel traffic through the deepened Connecting Channels and the St. Lawrence Seaway.

Connecting Channels Survey Report

General

The determination of economic justification for considered plans of improvement for deep-draft commercial navigation is represented by an estimate of transportation savings for prospective commerce as anticipated over the economic project life, general assumed as 50 years. These savings are measured by the difference between the transportation cost without the plan of improvement, that is with navigation conditions as now prevailing, and the transportation cost with the considered plan of improvement in being. The initial study involves an estimate of the bulk cargo vessel fleet which can be anticipated to develop over the economic life of the project, and an estimate of the fleet that would prevail without channel improvement. Then the related determination is that of the commerce to be anticipated to use the waterway over its economic life. With the respective fleets assumed to be in operation, the transportation cost per ton of commerce both without and with the proposed project is determined. The difference in total transportation cost thus derived represents the measure of transportation savings that can be credited toward the cost of improving the project.

Prospective Vessel Fleet

The United States bulk cargo vessel fleet at the time of preparation in 1953 of the Connecting Channels review survey report included 61 vessels with allowable loadline drafts between 22.4 and 26.9 feet. In order to analyze the economic justification of deepening the Connecting Channels, it was necessary to determine the general character of the fleet which could be anticipated with

the deepening project constructed. It was considered that the replacement of the small, old vessels with new, larger and faster vessels would be accelerated if deeper Connecting Channels were provided. Consultation by the District Engineer, Detroit, with vessel owners resulted in a prediction of the composition of the bulk cargo vessel fleet as of 1985, a time considered representative as average over the economic life of the project. An estimate was made of the composition of a fleet under conditions prevailing both without and with the Connecting Channels deepening project. These prospective vessel fleets are listed in Table 1. The two fleets of bulk cargo vessels, as then predicted, were used in determining the transportation savings for prospective commerce and which savings would be credited toward justification of the cost of deepening the Connecting Channels.

TABLE 1 - PROSPECTIVE UNITED STATES GREAT LAKES BULK CARGO FLEETS

		Typical Bulk Carriers		Typical Self-unloaders		Prospective Vessel Fleet in 1985			
						Without Connecting Channels-Harbors Improvement		With Connecting Channels-Harbors Improvement	
Class	Over-all Length (feet)	Draft (feet) (1)	Capacity (tons) (1)	Draft (feet) (1)	Capacity (tons) (1)	Bulk Carriers	Self-unloaders	Bulk Carriers	Self-unloaders
1	Under 400	-	-	-	-	0	0	0	0
2	400-499	21.0	10,000	-	-	2	0	2	0
3	500-549	21.3	12,900	22.2	11,800	5	4	5	3
4	550-599	21.9	15,500	21.7	14,000	74	18	28	6
5	600-649	25.0	22,100	25.6	20,200	139	7	55	3
6	650-699	25.7	23,300	25.8	22,400	2	1	58	7
7	Over 700	26.5	28,000	26.2	24,900	2	0	26	2
					Total	224	30	174	21

(1) Draft and capacity of vessels pertain to summer season.

Plan of Improvement

The design of the project for deepening the Connecting Channels is based on a bulk cargo vessel safe draft of 25.5 feet, giving appropriate allowances between vessel draft and channel depth to account for squat of vessel when underway, exposure of the individual channel reach to storm action, and character of channel bottom—whether hard or soft material.

The plan of improvement for deepening the Connecting Channels is based upon a design that will provide a safe vessel draft of 25.5 feet when the controlling lake is at its low water datum plane. Depths corresponding to this vessel draft for individual reaches of the Connecting Channels range from 27 to 30 feet. The maximum loadline limits of the existing and prospective vessel fleet were studied in determination of this advisable vessel draft for design purposes. In this study recognition was given to fluctuation of lake levels.

The fluctuation of lake levels must be recognized in the design of navigation projects on the Great Lakes and in the economic analysis of such considered projects. The plan for deepening the Connecting Channels includes remedial features which result in the deepening project having no affect on levels of the lakes. Under design conditions, a safe vessel draft of 25.5 feet

will prevail when the controlling lake level is at its low water datum plane. This datum plane is normally at an elevation such that the lake level seldom is lower than this plane. Seasonal variations in levels of each lake range between 1 and 2 feet, with seasonal low in winter and the high in summer months. However, the pattern of lake level fluctuations for each lake differs for range between high and low levels and in the time of occurrence of peaks and lows. It was determined that the level of Lake Erie with reference to its low water datum plane is generally higher than the levels of the upper Great Lakes with respect to their planes. A study of lake levels as they have fluctuated for the period since 1922, when Lake Superior came under complete control, indicates that a minimum depth of 27 feet throughout the Connecting Channels will prevail on the average for 96 per cent of the 8-month navigation season. For substantial periods of time when lake levels are above low water datum vessels with permissible loadline draft limitation can take advantage of available drafts in excess of 25.5 feet.

The allowances between vessel draft and channel depth as assumed in project design criteria will probably not be fully used by all vessels transmitting the Connecting Channels. Actual vessel drafts are for determination by vessel operators, since safe draft allowances vary with the characteristics of different types of vessels. In this connection, the Corps of Engineers furnishes to the Lake Carriers' Association prevailing lake levels on a continuing basis, together with lake levels anticipated during the ensuing month. With this information, and knowledge of actual vessel operating requirements, the Association issues recommended vessel drafts for those vessels which are in member fleets of the Association. These recommended vessel drafts take advantage of increments of draft as small as 1 inch. On six occasions in 1958 revised vessel draft recommendations were made for only 1-inch increment of change in vessel draft. The extent of draft utilization of the deepened Connecting Channels for ocean vessels using the St. Lawrence Seaway will be for determination by operators of those vessels.

Commerce

The derivation of prospective commerce for the next 50 years in iron ore, as made for the survey report on the Connecting Channels in 1954, considered iron ore reserves, including the Labrador and other Canadian sources, which would be available to meet the needs of the steel-producing industry; sources of low-grade iron ores for concentration and use in the steel industry; demand for iron ore and iron ore concentrates; and estimate of commerce in iron ore which would utilize the deepened Great Lakes Connecting Channels. In the Connecting Channels survey report it was estimated that anticipated total annual commerce in iron ore through the Connecting Channels over the economic life of the proposed improvement would be 82,000,000 net tons, of which about 65,000,000 net tons would be transported in vessels which would have permissible loadline limits such that advantage could be taken of the deepened Connecting Channels. Transportation savings on iron ore commerce credited to the plan for deepening the Connecting Channels were therefore based on this latter tonnage estimate.

Similar traffic analysis studies for stone and grain were also made for the Connecting Channels survey report. The majority of vessels used in these trades have loadline limits sufficient to utilize the deepened Connecting Channels. Detailed studies of prospective commerce in coal were not made

for the Connecting Channels survey report because the principal upbound channel used for coal commerce, the Amherstburg Channel in the Detroit River, had been previously authorized for deepening to a depth of 27 feet, which is more than ample for vessels in the coal trade.

Economic Analysis

The cost of the Connecting Channels deepening project included all work of deepening those channels which had not previously been authorized as work associated with the deepening project. Benefits recognized the various elements entering into the cost of transporting anticipated commerce over the estimated project life both without and with the Connecting Channels deepening. Evaluated benefits were for commerce using United States Great Lakes harbors and the Connecting Channels above Lake Erie. Benefits from prospective commerce through the St. Lawrence Seaway which would use the Connecting Channels were not evaluated for the purpose of this economic analysis. The survey report determined that the benefit-cost ratio for the Connecting Channels deepening is 1.78. As previously mentioned, this project was authorized for construction by Act of Congress approved 21 March 1956 and work in the Connecting Channels is now under way.

The economic analysis of the Connecting Channels deepening project included the cost of deepening the group of shipping harbors on the Upper Great Lakes and the receiving harbors on the Lower Great Lakes which would be needed to accommodate the traffic in larger vessels using the deepened Connecting Channels. Also, in undertaking the economic analysis of the Connecting Channels and harbors deepening, it has been recognized that a substantial portion of traffic in iron ore, stone, and grain will continue to be carried in smaller vessels to the smaller harbors or inner portions of the larger harbors, and would not benefit by deepening of the Connecting Channels. No transportation benefits were taken for such commerce.

Great Lakes Harbors Survey Report

Preparation of the Great Lakes Harbors report required more detailed analysis of prospective traffic because of need for distributing such prospective commerce between harbors which would utilize the deepened Connecting Channels. Further, the Great Lakes Harbors report needed to evaluate transportation savings to commerce using the St. Lawrence Seaway.

Vessel Fleet

Information available during preparation of the Great Lakes Harbors report indicated that vessel interests now propose construction of even larger and more economical vessels than had been contemplated during preparation of the Connecting Channels survey report and which would further enhance benefits computed for the Connecting Channels-Harbors deepening project. However, the previously determined prospective vessel fleets, without and with the Connecting Channels deepening project, were retained for economic analysis of individual harbors, as this assumption was considered conservative in view of the recent clear indication that the larger, more efficient vessels previously estimated are now being built. While the general character of the anticipated bulk cargo fleet as used in the Connecting Channels report was

retained insofar as distribution of vessels between the 7 classes based on vessel length, the recently constructed vessels in the larger classes indicated the advisability of a revision upward in average cargo capacity for representative vessels in the respective classes. Vessels in the largest class, overall length in excess of 700 feet, now are considered to have a representative capacity at midsummer loadline draft, 27.2 feet, of 28,900 tons. Representative class 7 vessels, over 700 feet long, are considered to carry average cargoes without and with the Connecting Channels-Harbors deepening project of 22,810 tons and 26,600 tons, respectively. For a representative class 5 vessel, 600 feet to 650 feet in length, the average cargoes are 19,540 and 21,680 tons under respective conditions. These averages represent cargoes carried with available drafts of 22.3 and 25.5 feet, with and with the project, respectively, when lake levels are at low water datum. These average cargoes also include consideration of fluctuation of lake levels and of variations in allowable loadline limits during the navigation season for individual vessel classes. It is seen from a comparison of these average vessel cargoes that the deepening project will result in more economies to the larger and newer vessels than to the smaller vessels because of the greater increase in carrying capacity of these vessels with the deepened channels. Even though such larger vessels have somewhat higher operation cost, the resulting unit cost of transportation per ton is less when the vessels can take advantage of their greater potential carrying capacity.

Commerce

As the Great Lakes Harbors report requires an estimate of potential commerce for each individual shipping and receiving harbor, a more thorough analysis of anticipated commerce in iron ore, stone and grain traffic was required for this report. The detailed re-analysis of iron ore traffic potential on the Great Lakes has been completed. It was determined that about 75 per cent of the national steel production capacity is in, or tributary to, the Great Lakes area; that the iron ore demand for this steel-producing industry for the estimated life of the proposed navigation project would require an annual average shipment of about 102,000,000 net tons of iron ore from harbors on the Great Lakes; and that the estimated annual receipts of iron ore at United States Great Lakes harbors should average about 138,000,000 net tons annually. The excess in receipts of iron ore at harbors would be met by traffic from Eastern Canada and other imported ores through the St. Lawrence Seaway. The portion of this prospective iron ore traffic which would move through the Connecting Channels is estimated to be 98,000,000 net tons, which is about 20 per cent greater than estimated movement as determined in 1954 for transit through the Connecting Channels. The revised estimate of the portion of traffic in iron ore through the Connecting Channels which would realize savings by use of the larger vessels is about 90 per cent of the total iron ore commerce through the Connecting Channels. This re-analysis also arrived at an estimate of iron ore tonnage, including iron ore through the Seaway, which would utilize individual shipping and receiving harbors.

Similar re-analysis studies were made of prospective commerce in stone, grain and coal. The stone traffic analysis is completed and the others are nearing completion. The Harbors report also requires that an estimate be made of prospective commerce through the St. Lawrence Seaway to and from harbors on the Great Lakes. Transportation savings credited to harbor

deepening for this commerce are included in the economic justification of deepening work at the harbors.

The general commodity cargo analysis of overseas traffic through the St. Lawrence Seaway to and from Great Lakes United States harbors is a complex task of considerable magnitude. Procedures used in this analysis, together with related considerations, are being covered in a separate paper and are not discussed herein.

Economic Analysis

The more detailed derivation of transportation savings as required in the study of need for improving individual harbors on the Great Lakes required that a detailed field survey be made of vessel operation cost for each class of vessels included in the United States bulk cargo fleet. This was done through contacts with owners and operators of Great Lakes vessels in the Spring of 1958. Representative annual vessel operation costs for typical vessels in the individual vessel classes were then determined and resolved to an hourly operating cost basis.

A further necessary step in the economic evaluation was study of the integrated movement of iron ore, stone, coal and grain by the United States bulk cargo fleet. All trips on the Great Lakes made by the United States bulk cargo fleet in 1957 were analyzed. The actual use of each vessel in its role of transporting these commodities was studied and the vessel trip time in 1957 chargeable to typical cargo movements was determined. For purposes of distributing vessel operation costs among the commodities carried, a general basic assumption was made that the light vessel movement from the last port of discharge to the loading port is chargeable to the next cargo movement. Modifications of this rule were used where a vessel made an extended side movement with a loaded backhaul when returning to an Upper Lakes port for loading the next cargo of iron ore. Centers of concentration representing harbors shipping and receiving these bulk commodities were utilized also. The 1957 vessel trip time which included travel and in-port time chargeable to commodity movements was adjusted to recognize that many vessels of the prospective fleet would be faster than at present.

This detailed analysis of the integrated trip movement by the United States bulk cargo fleet in the 1957 navigation season involved a review of about 10,000 individual trips of some 300 bulk carrier vessels. This detailed study, including information on time required and cargo carried, was possible as basic trip data were available through records compiled in the gathering of normal waterborne statistics for the Great Lakes. Effort and time put into analysis of basic data on trips and cargoes were well worthwhile, since it permitted a realistic approach in allocating vessel time to each cargo carried.

Transportation of iron ore is unquestionably the most important task of the Great Lakes bulk cargo fleet; nevertheless, traffic in other principal bulk commodities—coal, stone and grain—is also of such magnitude as to require a major effort by the fleet each navigation season. The economic potential in a well organized, integrated fleet movement to transport this variety of commerce is obvious.

The more detailed study for the economic analysis of the individual harbors in the Great Lakes Harbors study recognizes that bulk cargo traffic at specific harbors sometimes involves only a single lake or a combination of two of the Great Lakes rather than the entire Lakes system. The derivation of

average vessel cargoes without and with the considered improvements utilized lake level fluctuations which were relevant to the particular traffic movement involved. As a consequence, the adjustment of average cargoes due to lake level fluctuations differed appreciably when a particular movement involved less than the entire Great Lakes system as compared with a movement which used the entire system. The primary reason for difference in cargo under these circumstances is that Lake Superior is controlled and the extent of high level of that lake with reference to its low water datum plane is not as great as for corresponding conditions for the Lower Lakes.

The bulk cargo commerce and transportation cost derivation discussed herein are for Great Lakes intralake and interlake traffic. The derivation of transportation saving for traffic through the St. Lawrence Seaway to and from the United States Great Lakes harbors is developed in a similar manner. Except for iron ore, as mentioned earlier in this paper, and grain, interim reports completed to date on harbors have not included prospective Seaway commerce or evaluation of transportation savings to Seaway traffic. However, deepening of harbors for Great Lakes bulk cargo commerce will in most cases provide deeper channels which can be taken advantage of by Seaway traffic either with or without deepening work at other portions of the harbor.

Status of Study

The Great Lakes Harbors study is well under way and scheduled for completion in the Fall of 1960. The program for this comprehensive study is that interim reports be submitted initially on harbors where all necessary engineering and prospective commerce data could be developed in a relatively short time. Such harbors, or portions of harbors, are those for transport of iron ore, stone and grain and where conditions are such that the largest vessels can be used to take full advantage of the deeper depths under consideration. Submission of early interim reports does not preclude subsequent submission of interim reports on improvements at separate harbors, or even on additional improvements at harbors previously reported on. Subsequent interim reports on harbors will include those for which justification is dependent primarily upon transportation savings to general cargo commodities using the St. Lawrence Seaway. There have been 16 interim harbor project reports submitted by the Division Engineer. About 50 harbors are under study in the Great Lakes Harbors report. This report will incorporate the findings of the interim reports in its summarization of the integrated harbor system.

CONCLUSIONS

The evaluation of benefits for the system project for deepening the Connecting Channels-Harbors has been an extensive task, but it has also been a most rewarding endeavor. The adaptation of basic theories to newer methods and refinements of analysis all tend to strengthen project justification as used in navigation studies. Perhaps more than anything else, it has also clearly demonstrated that the effectiveness of the Great Lakes deep-draft navigation system is interdependent on the characteristics of the using fleet, the operation of the fleet, and the makeup of navigation facilities in the system. Each of these elements can seriously limit overall capabilities; all are essential in achieving maximum utilization and economy.

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METHODS OF CORRECTING WAVE PROBLEMS IN HARBORS

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SYNOPSIS

The causes of and some of the cures for objectionable wave conditions in harbors are discussed. The use of hydraulic models to determine methods of reducing wave action is illustrated by means of examples. Various methods of solving the wave problems which existed in four harbors are described.

INTRODUCTION

A harbor may be defined as "a water area nearly surrounded by land, sea walls, breakwaters, or artificial dikes, forming a safe anchorage for ships."(1) Webster gives the following shorter definition: "A place of security and comfort; a refuge." The concept of safety or security implies that the harbor area is shielded from the violence of major storms sufficiently to prevent the destruction of anchored ships and harbor facilities. If the barriers are made sufficiently strong to prevent failure as the result of wave forces and high enough to prevent serious overtopping, this objective can be achieved. It is the requirement for "comfort" which is less easily attained. Because one or more openings sufficiently large for the safe passage of ships must be provided, some wave energy enters the harbor area no matter how effective the breakwater arrangement may be. As the waves enter the harbor opening, diffraction causes them to travel forward over a fan shaped area. Where relatively smooth vertical sea walls are encountered within the harbor, the entering waves are often reflected several times with only a small amount of energy loss. The combined effect of diffraction and reflection may project wave action into remote portions of a harbor which appear to be completely sheltered. Under certain conditions the wave heights within the harbor are

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larger than those of the entering waves, either because of resonance or because the wave energy may be focused on particular locations. Eventually, of course, the wave energy will be dissipated because of turbulence and friction. However, the constant input of energy through the harbor entrance may maintain sufficient wave action to make mooring at docks difficult or unsafe and to make the motion of small boats so violent that it is very uncomfortable to stay aboard and impossible to carry on maintenance or repairs. The wave pattern is exceedingly complex in most harbors with the result that analytical methods or even many years of experience are often of little assistance in planning remedial measures. However, the hydraulic model provides an analogue by means of which the most effective solution can be determined for any harbor wave problem. By means of model studies, barriers, wave absorbers or wave diffusers can be designed and located in such a manner as to provide the maximum reduction in wave height at the lowest cost.

Causes of Wave Motion in Harbors

Although some wind waves are generated within a harbor area, most of the objectionable wave action is created by the entrance of wave energy through harbor openings. These waves are dispersed with varying intensity throughout the harbor area by diffraction or reflection. Most of the entering waves are typical wind-generated waves having periods varying from 4 to 20 seconds. However, long-period waves,(2) created at distant points on the oceans or resulting from seiches on large lakes, may create undesirable conditions in harbors. Waves of this type may have periods varying from one minute to as long as 60 minutes. They may be created by seismic disturbances, or barometric pressure fluctuations, but it is believed that most of them may result from the combination of wave trains of slightly different periods. This phenomena also creates the long-period pulsations called "surf beats."(2,3) No matter how the waves were created, their effect on a harbor consists of introducing energy periodically. Because waves are reflected particularly from vertical walls with little loss in energy, the combined effect of diffraction and reflection is to spread the energy throughout much of the harbor area. Waves in harbors are oscillatory waves which are characterized by an orbital motion of the water. The water particles rotate in elliptical paths having relatively long horizontal axes and shorter vertical axes. When two or more waves are superimposed, the water motion is the approximate resultant of the various components of motion. Thus, when two crests coincide, the resultant wave height will be approximately the sum of the two individual heights. Similarly, the coincidence of a trough of one wave with the crest of another would cancel out all or part of the vertical motion at that point, but the horizontal amplitudes and velocities would reinforce each other. Thus, when a wave strikes a vertical wall at a 90° angle and reflects straight back, a standing wave pattern is formed as illustrated schematically in Fig. 1. In this figure it is assumed that waves of length L and height H approach from the right and are reflected from the vertical wall at the left. After the first incident wave reflects from the wall and travels one-half wave length to the right, it encounters the following incident crest. Because of the addition of the two vertical components of motion a wave of height 2H is produced at this location. Furthermore, the horizontal motions are equal and opposite and thus the horizontal velocity is reduced to zero. This situation is then

repeated every half wave length as the reflected waves travel seaward. These regions of maximum vertical displacements may be called antinodes[4] and are designated as A in Fig. 1.

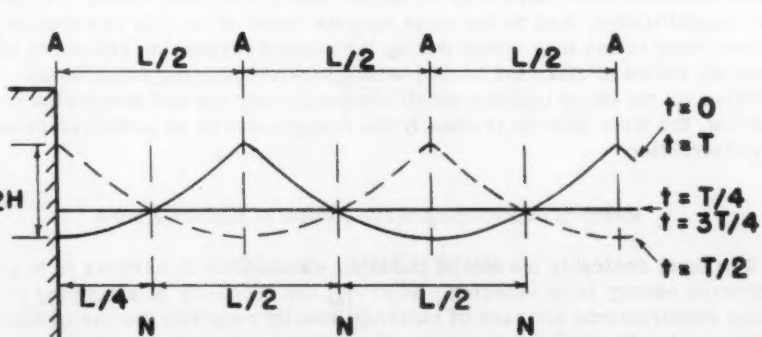
Similarly, at a point $\frac{L}{4}$ from the wall the first reflected wave encounters an oncoming trough and the vertical components of displacement are cancelled but the horizontal components of velocity are both toward the right and thus combine to produce a horizontal velocity twice as great as that for a single wave. Such locations of zero vertical displacement and maximum horizontal displacement may be called nodes and are designated as points N in Fig. 1.

The variations in wave configuration are cyclic, passing through one complete cycle during each wave period T. As illustrated in Fig. 1, if the solid line indicates the water surface at zero time, then at time $t = \frac{T}{4}$ the water surface is a horizontal plane, at $t = \frac{T}{2}$ the dotted lines indicate the water surface, at $t = \frac{3T}{4}$, the plane surface is repeated and at $t = T$ conditions are the same as at $t = 0$.

The wave motion illustrated in Fig. 1 demonstrates the possibility of establishing resonant conditions in a harbor. For example, a vertical wall placed at one of the points A in Fig. 1 would not interfere with the wave motion and waves of high amplitude could be established and maintained if wave energy in phase with the oscillations is being supplied. In general, any harbor area bounded by two parallel vertical walls would be subject to resonance if the distance between walls were

$$\ell = \frac{nL}{2} = \frac{nCT}{2} \tag{1}$$

where n is any integer.[5] The wave period is established by the waves entering the harbor, but C is a function of the depth within the harbor. The general expression for C is



REFLECTED WAVES

Fig. 1

$$C = \sqrt{\frac{gL}{2\pi} \tanh \frac{2\pi d}{L}} \qquad (2)$$

However, for the depths and wave lengths ordinarily encountered in harbors an approximate value of C may be found from the following equation

$$C = \sqrt{gd} \qquad (3)$$

Any distance (ℓ) between vertical walls, other than those which satisfy Eq. (1), would produce some damping and the maximum damping action would occur for

$$\ell = \frac{mL}{4} \qquad (4)$$

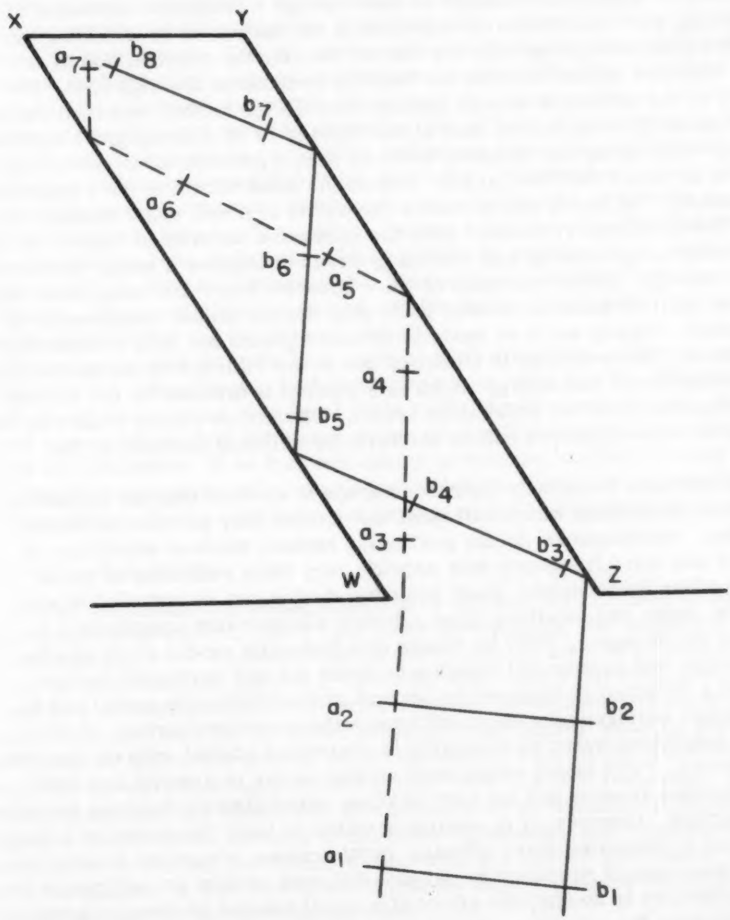
where m is any odd integer. This is because walls separated by distance in accordance with Eq. (4) (located at points N in Fig. 1) would stop all particle motion at these locations.

In addition to the possibility of resonance, illustrated by the simplified example above, waves may be reflected in such a manner as to focus wave energy at particular locations. This is illustrated by the example shown in Fig. 2. Here waves having crests $a_1 - b_1$ approach the slip WXYZ. Two selected orthogonals (lines perpendicular to wave crests), designated as $a_1 - a_2$ and $b_1 - b_2$, are projected into the slip by assuming that the angle of reflection is equal to the angle of incidence.[4] The orthogonal $b_1 - b_2$ progresses into the far corner of the slip X by means of 3 reflections. The orthogonal $a_1 - a_2$ is deflected slightly to simulate refraction at W and reaches the vicinity of X after only two reflections. Successive wave crests along these orthogonals are shown as a_1, a_2, etc. and b_1, b_2, etc. The crests a_7 and b_8 reach the vicinity of X at about the same time, thus illustrating one manner in which the energy from two separate portions of a wave crest may be concentrated at one location. The wave height at that location would be approximately the sum of the two original wave heights.

The foregoing examples have been presented to show how wave heights at some locations in harbors may be larger than the incident waves. Although such amplification may be the most serious cause of trouble in a harbor, sufficient wave action may result during the normal dispersion and decay of the entering waves to make the harbor unsuitable for mooring small boats. Whether or not there is some amplification or only normal dispersion is occurring, the wave pattern is usually too complicated to be predicted in advance of construction.

Methods of Reducing Wave Action in Harbor Areas

The most desirable method of reducing wave action in harbors is to prevent wave energy from entering. However, the necessity of satisfying navigation requirements for ease of entrance usually requires the use of harbor openings of sufficient size to permit the entrance of a considerable amount of wave energy. When new harbors are planned it is often possible to make an arrangement whereby nearly complete protection is provided against waves from the prevailing wind direction and a beach or other type of wave absorber is placed so that the energy of the waves which do enter the harbor is



WAVE FOCUSING

Fig. 2

dissipated before reaching the mooring area. However, this paper is concerned principally with reduction of waves in existing harbor areas. This can be accomplished by building additional offshore breakwaters, by installing devices to impede the passage of wave energy through the entrance, by constructing wave absorbers or diffusers in the harbor or by shielding particularly vulnerable areas with sea walls. Usually the construction of additional offshore breakwaters is not feasible because of the high cost. Restriction of the amount of energy passing through the harbor entrance can be accomplished by reducing the size of the opening or by placing groin systems, or zigzag walls along the entrance walls in such a position as to reduce the reflection of waves into the harbor. One of the most effective wave absorbers is a relatively flat beach which causes the waves to break some distance from shore. The frictional resistance plus the excessive turbulence caused by collisions between up-rushing and returning waves dissipates a major portion of the wave energy. Other examples of wave absorbers are low walls over which waves can spill or walls so located as to stop the horizontal components of wave motion. Zigzag walls or systems of short groins act both as absorbers and diffusers. Some energy is absorbed due to the violence of the motion as waves collide while that energy which is reflected is diffused by the variety of directions in which the reflections travel. Oftentimes zigzag walls can be used both as wave diffusers and as shelters for selected portions of the harbor.

Conditions vary so greatly from one harbor to another that the method or combination of methods which will quiet one harbor may provide no benefit for another. Furthermore, in any particular harbor, devices which are effective for one wave frequency may provide very little reduction in wave height for other frequencies. Each remedial device can be installed in various shapes, sizes and locations, thus creating a hopelessly complicated dilemma for the designer. Only by means of a hydraulic model study can the most effective and economical solution be found for any particular harbor.

Built at a sufficiently large scale, an undistorted hydraulic model can be a near perfect analogue of wave conditions in the prototype harbor. Under prototype conditions water wave motion is controlled almost entirely by gravitational forces. Very small waves such as may occur in a model are influenced by surface tension and for very shallow water viscous damping becomes more important. However, it is usually possible to build the model at a large enough scale to minimize these effects. Furthermore, where the relative reduction in wave height produced by several different harbor arrangements or corrective devices is sought, the effect of a small amount of viscous damping will not influence the selection of the best design.

The model must include, not only the harbor area itself, but sufficient offshore area so that water will be deep enough for the operation of the wave generator. As waves leave deep water and enter water having depths less than one-half wave length, their length, velocity and orientation are affected by the bottom topography. Usually limitations on the size of the model make it necessary to locate the wave generator at a depth less than L/2. Therefore, the size and orientation of the waves of this location must be computed from deep water waves by means of refraction diagrams. All prototype wave characteristics are reproduced to scale in the model. Probably the most important of these is the wave period. Because waves are controlled primarily by gravity, the Froude model law applies and the wave period in the model is the prototype period divided by the square root of scale ratio. Actual records

of wave heights are seldom available; therefore wave heights and wave periods must be computed from wind records(6) for selected wind directions. Having determined the characteristics of typical model storm waves, the model is placed in operation and a series of observations are made to determine whether the objectionable prototype wave characteristics are being accurately reproduced. Usually, minor adjustments are required to produce a perfect analogy of the original harbor conditions. After the model is considered to be operating in a satisfactory manner, the process of evaluating various corrective measures is started. One method of proceeding is to test a number of arrangements or devices rather quickly until one is found which produces a noticeable reduction in wave height. This device is then tested more thoroughly by obtaining continuous records of wave heights at selected harbor locations, both for the original harbor condition and after the device being tested has been installed. The reduction in wave height at each gage location and the average reduction for the entire harbor may then be determined. The most effective device can be selected by comparison of the results obtained for all the devices tested.

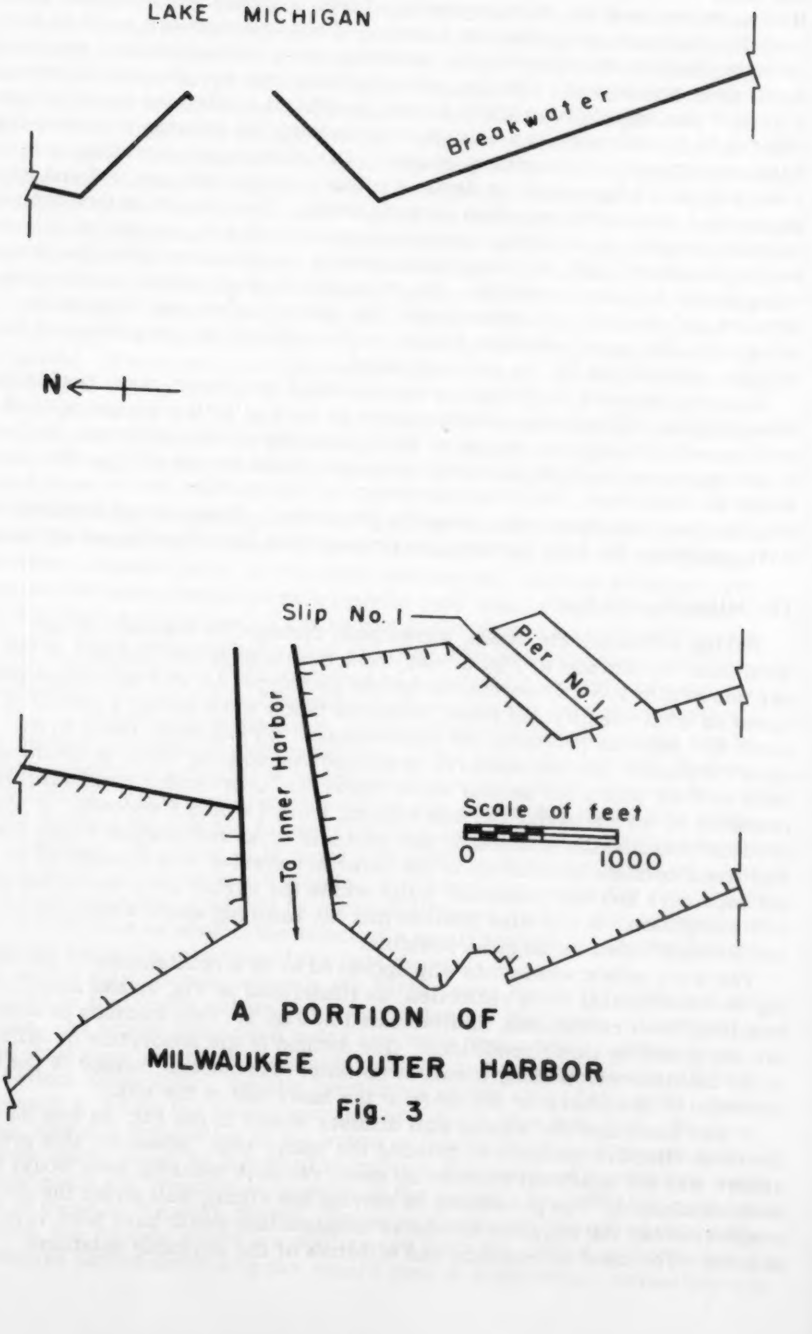
Each device must be tested for various wind directions, wave periods and wave heights. Oftentimes a device which is very effective for one wind direction and wave period may be of little value for other conditions. In fact, it is not uncommon to find situations where no single device will be effective under all conditions. It is then necessary to incorporate two or more devices into the final plan to provide complete protection. Examples of solutions to wave problems for four harbors are presented in the following paragraphs.

The Milwaukee Harbor

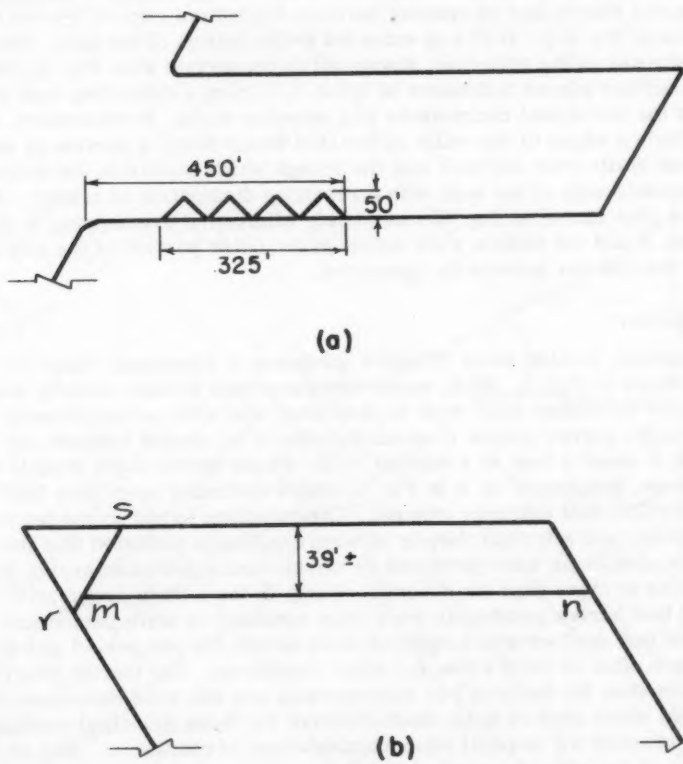
During northeasterly winds, waves pass through the entrance to the Milwaukee breakwaters, Fig. 3, and cause overtopping and damage at the inner end of Slip 1.(7) Observations on the prototype, as well as computations based on wind velocity and fetch, indicated that a wave having a period of about five seconds produces the most damage. Preliminary tests on the model indicated that the observed prototype overtopping could be produced most exactly with a 5.5 second wave. However, overtopping could also be produced by waves having periods varying from 4.8 to 6.5 seconds. It was necessary to find the solution to this problem by an installation within the slip itself because an addition to the outer breakwater was considered to be too expensive and any additional walls within the harbor area would interfere with navigation. It was also desired that the mooring space within the slip itself be maintained as large as possible.

The wave action within this slip appeared to be a combination of the standing waves resulting from reflection, as illustrated in Fig. 1, and amplification resulting from reflections, as illustrated in Fig. 2. Two methods of solution are suggested by these conditions. One method is the absorption or diffusion of the incident waves along a wall of the slip. The second method is the construction of absorbers or diffusers at the inner end of the slip.

It was found that the zigzag wall diffuser shown in the Fig. 4a was one of the most effective methods of quieting the entire slip. However, this procedure was not practical because so much valuable mooring area would have been eliminated. The possibility of moving the zigzag wall under the dock was considered but the required extensive construction would have been very expensive. The most economical and effective of the workable solutions



A PORTION OF
MILWAUKEE OUTER HARBOR
Fig. 3



CORRECTIVE METHODS

MILWAUKEE SLIP NO. 1

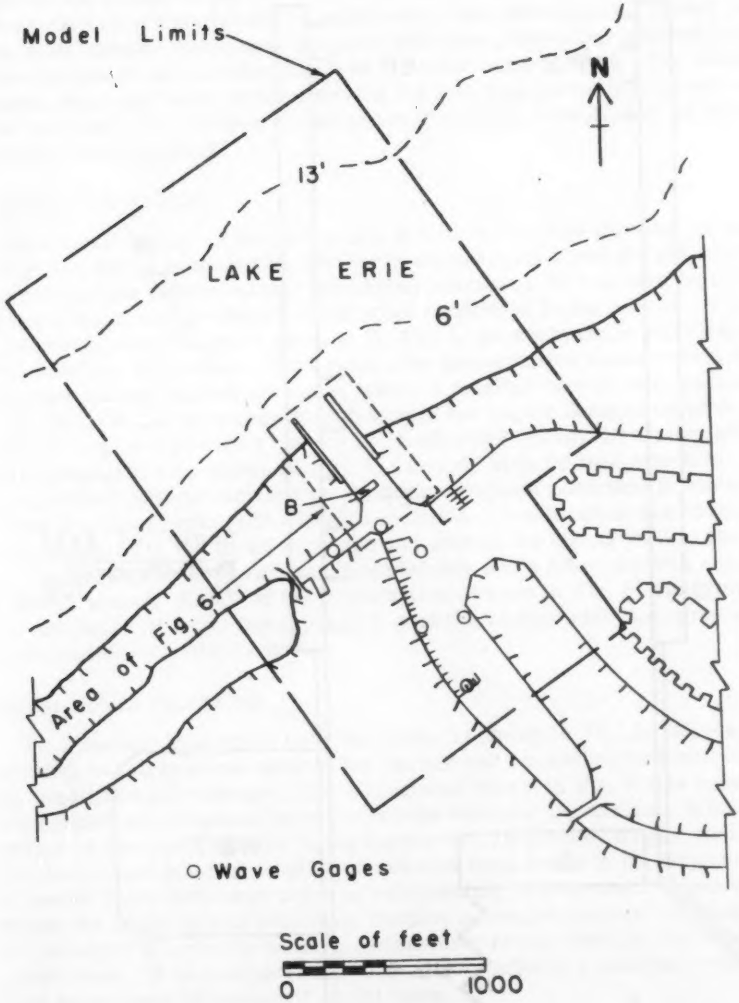
Fig. 4

consisted of walls located near the end of the slip having their top edges at the elevation of the water surface. One such solution is shown in Fig. 4b. The upper edges of both walls shown in Fig. 4b were at the water surface elevation. The wall m-n extended only eleven feet below the water surface, leaving about eleven feet of opening between the bottom edge of the wall and the bottom of the slip. Wall r-s extended to the bottom of the slip. These walls make use of the principle, discussed in connection with Fig. 1, that a vertical surface placed a distance of about L/4 from a reflecting wall will damp out the horizontal components of a standing wave. Furthermore, by placing the top edges of the walls at the still water level, a portion of each wave crest spills over the wall into the trough which occurs at the same time on the opposite side of the wall with a resulting dissipation of energy. Although the plan shown in Fig. 4b completely eliminated overtopping at the end of the slip, it did not reduce wave action in the outer portion of the slip as much as the diffuser previously discussed.

Mentor Harbor

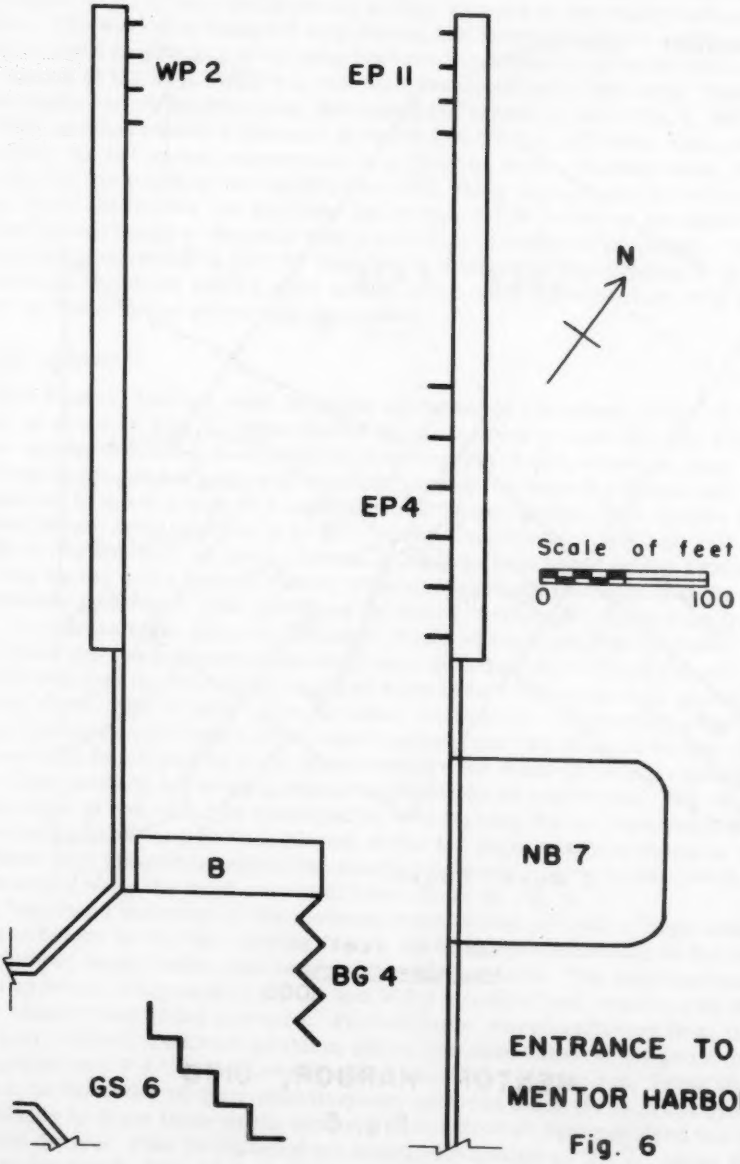
This harbor, located about 20 miles northeast of Cleveland, Ohio, on Lake Erie, is shown in Fig. 5. When waves are generated in Lake Erie by winds in the sector extending from west to northwest, the wave action in many portions of the harbor makes it uncomfortable to be aboard vessels and dangerous to moor a boat at a dock or wall. These waves enter despite the sunken barge, designated as B in Fig. 5, which obstructs more than half the width of the 200-foot entrance channel. Computations based on the fetches for this sector and a typical variety of wind conditions indicated that the objectionable conditions were produced by waves having periods varying from four seconds to more than six seconds. Early in the testing program[8] it was found that harbor conditions were very sensitive to wave period and direction and that devices which reduced wave action for one period and direction were often of little value for other conditions. The testing program was organized on the basis of two wave periods and two wind directions, but the methods which proved to be most effective for these principal conditions were also checked for several other combinations of conditions. The effectiveness of any plan was evaluated by determining the average reduction in wave height (over original conditions) at the six gage locations shown in Fig. 5. More than 200 combinations of remedial measures were tested and the three which were the most successful are shown in Fig. 6.

It was found that none of the methods tested would provide a large reduction in wave height for waves entering from a direction perpendicular to the shore line without some restriction near the tips of the piers. The combination of groin systems designated as EP 11 and WP 2 gave the best results with the least constriction of the entrance. Furthermore, waves reflected from the east pier created a difficult problem which was eliminated by the groin group designated as EP 4. Consequently, all three of the better plans, Plans 50, 53, and 51, in the order of their effectiveness, included these three groin systems. In addition to these three groin systems each effective plan required one additional device. Plan 50 included the beach, designated as NB 7. As an alternate to the beach, Plan 53 included the zigzag wall, designated as GS 6, whereas for Plan 51 the alternate was the wall BG 4. The average reductions in wave height for the three plans were approximately 78, 73, and 65 per cent, respectively. Among the 200 or more plans investigated were some with only



MENTOR HARBOR, OHIO

Fig. 5



ENTRANCE TO MENTOR HARBOR
Fig. 6

small variations from the ones illustrated here. Examples would be the addition or elimination of one or more groins in a groin system, a change in the length of the groins or the width of the beach or the length of the zigzag walls. Changes in the orientation of the walls were also investigated. In this harbor, as in most similar situations, the most effective plans often differed only a minor amount from considerably less effective combinations. The recommended plans are those which provided the best results for the lowest construction cost. Any further improvements could be attained only at relatively large increases in cost.

Rochester Yacht Club

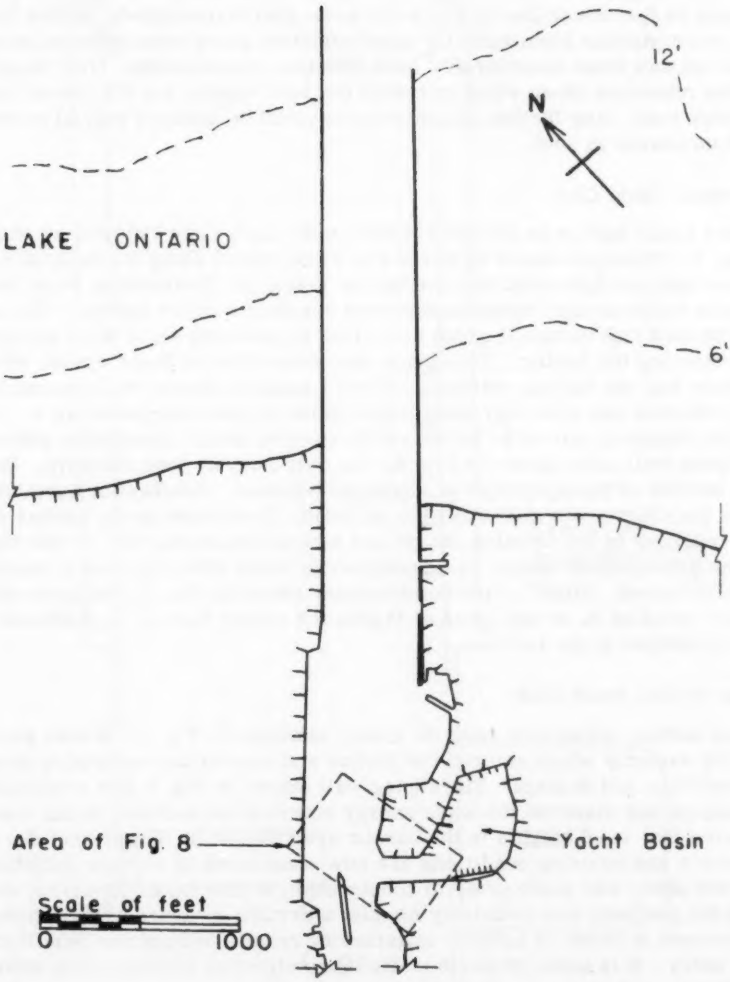
This small harbor is located nearly a mile from Lake Ontario, as shown in Fig. 7. Waves produced by northerly winds travel along the navigation channel and are diffracted into the harbor entrance. Reflections from the entrance walls create objectionable wave conditions in the harbor. The model tests showed that the short groin at D, Fig. 8, prevents some wave energy from entering the harbor. This groin also concentrates those waves, which still turn into the harbor entrance, within a smaller sector, thus permitting a more efficient use of energy dissipaters in the region designated as A. A beach in region A proved to be the most effective energy dissipater although the zigzag wall, also shown in Fig. 8, was only slightly less effective. However, neither of these devices produced an adequate reduction in wave height unless the short groin at D was also included. Conditions in the harbor were quite sensitive to the location, shape and size of the zigzag wall in the region A. The arrangement shown was considerably more effective than a number of others tested. Either of the combinations shown in Fig. 8, the groin at D plus the beach at A, or the groin at D plus the zigzag wall at A, produced very quiet conditions in the harbor.

Grosse Pointe Yacht Club

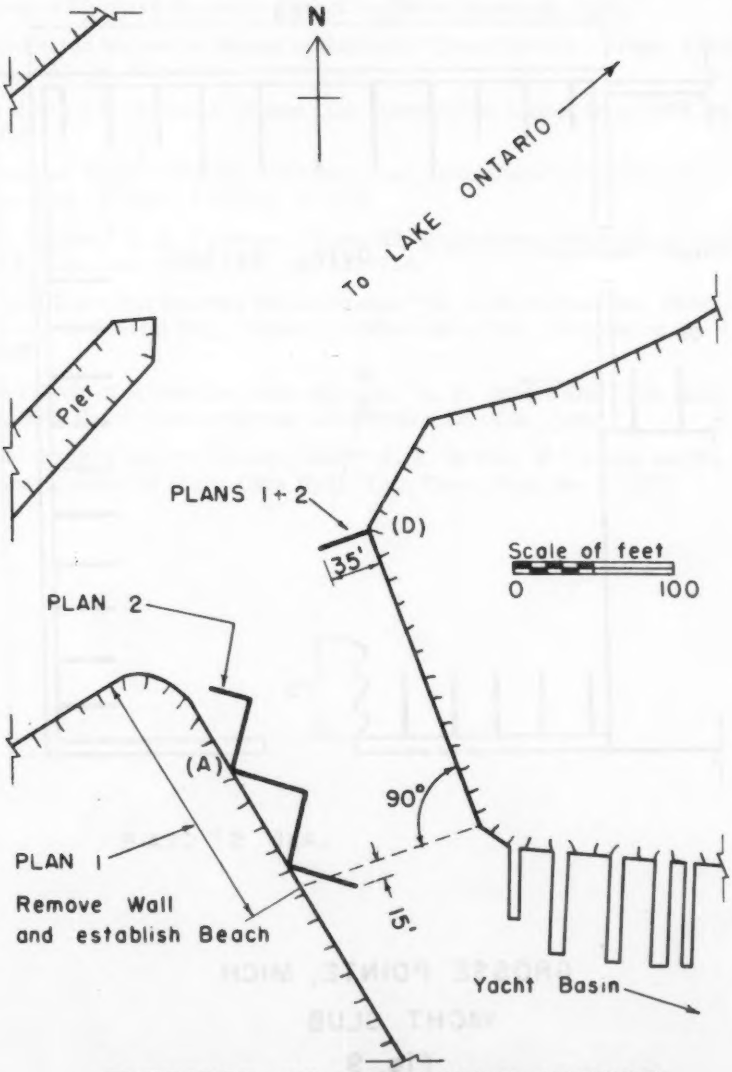
This harbor, located on Lake St. Clair, is shown in Fig. 9. Waves produced by easterly winds entered the harbor and caused uncomfortable mooring conditions and damage. The zigzag wall shown in Fig. 9 was constructed to dissipate and disperse the wave energy entering the harbor. It has been estimated that wave heights in the harbor are reduced by 60 per cent due to this device and mooring conditions are now considered to be very satisfactory. No model study was made prior to construction in this case. However, even though the problem was relatively simple, a certain measure of good fortune was involved in order to achieve satisfactory results without the benefit of a model study. It is quite probable that a more effective solution could have been determined by means of model tests.

SUMMARY

Harbor entrances admit not only boats, but also some waves. The amount of wave energy entering the harbor may be sufficient to cause damage to moored vessels and harbor facilities. In some cases resonance of reflected waves or the focusing of wave energy creates waves which are larger than those entering the harbor. Effective methods of reducing wave action in the harbor can be best determined by means of model studies. The wave reduction is accomplished by installing wave absorbers or wave dissipaters.

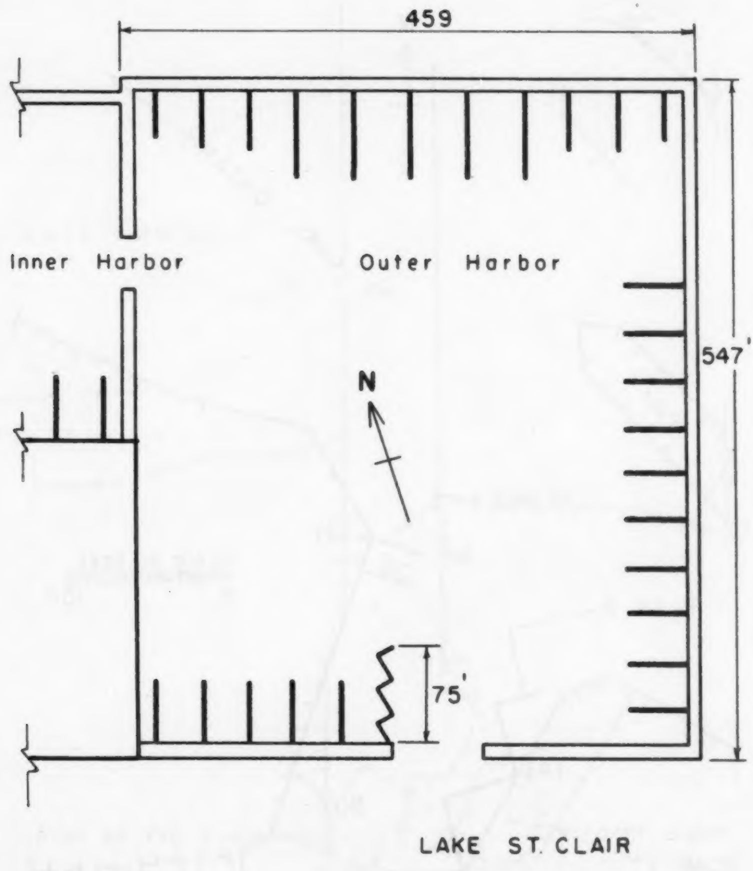


ROCHESTER, N.Y.
YACHT CLUB
Fig. 7



ENTRANCE TO ROCHESTER HARBOR

Fig. 8



GROSSE POINTE, MICH.
YACHT CLUB
Fig. 9

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AN APPROACH TO THE STUDY OF OVERSEAS TRAFFIC POTENTIALS[a]

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ABSTRACT

This paper deals with the methodology for estimating traffic potentials of general caro in the United States foreign trade. Comparisons are made of existing commerce into and out of the Great Lakes area through available commerce routes of the past and new commerce routes resulting from the opening of the St. Lawrence Seaway.

INTRODUCTION

This paper deals with a methodology for estimating traffic potentials of general cargo in United States foreign trade. Such methodology can be used in measuring the impact of deep-draft navigation on overseas commerce into and out of the Great Lakes area through the Connecting Channels on the Great Lakes and the St. Lawrence Seaway. Based on a nation-wide study of where exports originate and where imports terminate in the United States, these methods may be the key to greater knowledge of transportation economics in foreign trade.

The purpose of this paper, therefore, is to explain in general terms basic problems which had to be resolved together with possible solutions. In so doing it is pointed out that any formulas or methodology proposed herein but not yet accepted by the U. S. Army Corps of Engineers represent the views of the author and not necessarily those of the Corps.

One particular aspect of the study will be emphasized above others to illustrate methodology for measuring the volume of general cargo tonnage expected to flow through the ports on the Great Lakes via Connecting Channels and the St. Lawrence Seaway. With respect to such general commodity overseas

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a. Presented at the May, 1958, Convention in Cleveland, Ohio.

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traffic the main consideration of this paper will center around means by which it may be possible to gauge with reasonable accuracy the nature and potentials of the export traffic originating and import traffic terminating in an area tributary to the ports on the Great Lakes.

To place the problem of estimating Great Lakes/Overseas traffic potentials in proper perspective it should be pointed out first, why the U. S. Army Corps of Engineers is undertaking the study of potential overseas traffic through Great Lakes ports, and second, why information on traffic potentials is important, whether considering future traffic through the Great Lakes, the Connecting Channels and the Seaway, or through seaboard ports on the Atlantic, Gulf and Pacific coasts of the United States.

Authority for the Traffic Study. A Great Lakes Harbors Study was authorized by resolutions by the Committee on Public Works, United States Senate, adopted 18 May 1956 and by the Committee on Public Works, House of Prepresentatives, United States, adopted 27 June 1956. These resolutions requested the Board of Engineers for Rivers and Harbors to review the reports of the Chief of Engineers on the Great Lakes Connecting Channels with a view to determining the advisability of further improvements of the harbors on the Great Lakes in the interest of present and prospective deep-draft commerce, with due regard to the scheduled time of completion of the St. Lawrence Seaway and the Connecting Channels between the Great Lakes.

Preparation of such a comprehensive study includes an evaluation of potential overseas traffic through the Great Lakes ports. The final overseas traffic report is nearing completion. This paper deals principally with the methodology of applicable research.

The Six Phases of the Traffic Study. Basic procedures of a Great Lakes-Overseas traffic study suggest an arrangement of its several major phases so that development of the methodology can be more readily followed.

Six such phases of a traffic study of this nature are:

I. General Approach to the Study
Development of methodology and mechanics of determining points of origin and destination of commodities in U. S. foreign trade.

II. Origin/Destination Phase
Analysis of points at which exports and imports enter the flow of U. S. overseas foreign trade.

III. Base Period Phase: 1955-1956
Study of U. S. production, consumption, exports and imports past and present to establish base from which to estimate future traffic.

IV. Projection Phase: 1960-2010
Consideration of economic growth prospects for United States in general and Great Lakes tributary area in particular, in order to estimate potential traffic over the 50-year period, 1960-2010.

V. Transportation Analysis Phase: 1958
Study of recent overland and ocean operating costs and freight rates to establish cost and rate differentials for routings via Great Lakes ports versus routings via seaboard ports.

VI. Potential Great Lakes Traffic Phase: 1960-2010
Evaluation of factors considered in Sections I to V so as to indicate level of future traffic expected to be generated in the Great Lakes tributary area and oriented toward the Great Lakes, and allocation of that potential traffic to individual harbors on the Great Lakes.

Each of these six phases is important by itself, but even more significant when viewed as a combined research effort in estimating Great Lakes and Seaway traffic potentials.

I. General Approach to the Study

During recent years many estimates of the foreign overseas traffic potentials of the Great Lakes and the St. Lawrence Seaway have been made. All, of necessity, had to be made without benefit of information now available as to where exports originated or imports terminated within the United States.

Defining the Great Lakes Area. The Great Lakes Tributary Area can be defined in several ways all varying as to the sphere of influence extending into the hinterland. Of prime importance, of course, is the heavy concentration of traffic generated in the immediate vicinity of the port cities themselves. In the ever widening circles the hinterland areas of each port reach out 100 miles, 250 miles, 500 miles and so on until they merge. For the purposes of this paper, and until the final results of the traffic study show which are the most economical routings, an area which conforms roughly to the sphere of influence of the Great Lakes ports can be considered as including the western portions of New York and Pennsylvania; and most of the Mid-West states such as Ohio, Indiana, Michigan, Illinois, Wisconsin, Minnesota, Iowa, Missouri, North Dakota, South Dakota, Nebraska and Kansas.

This area, represents approximately one-quarter of the land area of continental United States and one-third of the total population. Within its boundaries is about one-third of the nation's standard metropolitan areas (a group of contiguous counties containing at least one city of 50,000 or more inhabitants). This area also accounts for more than one-third of the dollar value of the nation's manufacturing and almost one-half of the dollar value of its agriculture.

Most of this activity of the area is less than 200 miles from any port on the Great Lakes. So until a final delineation of the Great Lakes tributary is determined, this region will serve as a reasonable approximation of the area tributary to the Great Lakes. Within the limits of the area tributary to the Great Lakes, however finally defined, exports begin their overseas journey and imports reach a destination in someone's factory, warehouse, store, shop, or home.

The delimitation of the Great Lakes tributary area will be determined largely through development of transportation differential[1] data which will indicate whether shipments to or from a particular point in the United States are oriented toward a Great Lakes port or toward a seaboard port.

Problems of the Traffic Study. The problems in a study of this nature generally are threefold: first, how to determine where exports originate and where imports terminate within the Great Lakes Tributary Area; second, how to measure the magnitude of potential Great Lakes-Seaway foreign trade; and third, how to define the extent of the Great Lakes Tributary Area. In the early consideration of these questions it became quite evident that no complete data were available on a nation-wide scale with respect to:

1. How far away from the port areas in the United States overseas traffic originates or terminates.

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1. Transportation differentials can be figured on the basis of freight rates or carrier costs; both of which are being considered in the Great Lakes Traffic Study.

2. How much traffic is carried by railroads, by trucks, or over inland waterways.

3. In what states do traffic origins and destinations tend to concentrate.

4. What export-import commodities are predominant in the Great Lakes area.

5. What principal foreign areas exchange their products with the Great Lakes tributary area.

6. Through which U. S. coastal ports are most of the Great Lakes tributary area commodities shipped.

With data now available and pertinent to these and many other questions, it was considered practicable to estimate traffic potentials of the Great Lakes area without having to use assumed points of origin and destination from which to calculate whether it costs less to ship via a Great Lakes port and the Seaway than overland through a Seaboard port.

II. The Origin/Destination Study

Cooperative efforts of the Bureau of the Census were most helpful in developing methods and procedures which for brevity may be referred to as the Origin/Destination Study.

The Origin/Destination study is proving to be a most important key to many answers. It consists primarily of a sampling process applied to official records of U. S. foreign trade data together with questionnaires sent to each of 15,000 shippers to find out the actual point at which their export or import commodities entered U. S. foreign trade.

Sampling Techniques. Since it was physically impossible from the standpoint of time and money to analyze all of the more than 2,000,000 export and import transactions which constitute United States foreign trade each year, it was necessary to exclude certain categories of commodities (mostly bulk items) from consideration and to employ a sampling method for the remaining general cargo commodities. Sampling techniques developed by Dr. D. E. Church, Chief, Transportation Division of the Bureau of the Census were designed to produce the greatest degree of accuracy for the largest volume of exports and imports to be studied. The size of the final sample was, of course, determined by the funds available.

Selection of Commodities. The "universe" from which the sample was taken consisted of a deck of approximately 2,160,000 foreign trade (waterborne) punch cards containing data from import entries and export declarations filed for the calendar year 1956. The import deck contained about 660,000 cards and the export deck about 1,5000,000 cards. "Special categories" and shipments below the weight and dollar cut-off for the Bureau's foreign trade tabulations were not included.

Preliminary to selecting a "gross sample" from the "universe" of export and import cards noted above various bulk commodities such as iron ore, limestone, coal, grains, certain minerals and bulk petroleum, for which special studies were in preparation, were removed from the basic "universe." The object was to obtain a sample representative of "liner type" cargo rather than "tramp type" cargo.

In devising procedures for drawing the gross sample it was assumed that any bulk commodity shipments remaining in the universe would move in large units, probably in excess of 100,000 pounds. This assumption, upheld in subsequent tests, became the basis for a stratification system.

The gross sample, for exports and imports, separately, was selected from strata based on pounds. This stratification by weight of shipment, and withdrawal rate for each strata showed:

Size of Strata		Ratio of cards to be withdrawn from "universe"
Strata 1, - 1,000,000 pounds and over	-	all cards
Strata 2, - 100,000 - 999,999 pounds	-	1 card in 10
Strata 3, - 10,000 - 99,999 pounds	-	1 card in 50
Strata 4, - under 10,000 pounds	-	1 card in 250

The gross sample withdrawal rate reduced the original universe deck from 2,160,000 cards to approximately 120,000 cards, half exports, half imports. While representing only 5 to 6 percent of the number of card transactions the gross sample accounted for more than 90 percent of the total weight of shipments in the universe.

Prior to selection of the net sample, specific commodities in addition to those omitted from the "universe," were deleted from the gross sample also. These additional deletions from the export deck included petroleum (oils), cotton (unmanufactured), grains, soybeans and flaxseed, and sulphur, and from the import deck—petroleum (oils), bananas, sugar and pulpwood. The final number of transactions in the net sample was approximately 7500 export items and 7500 import items. It represented about 15 percent of the number of card transactions in the gross sample and accounted for about 40 percent of the weight of all shipments in the gross sample.

The next step was to withdraw from Census files and examine the respective documents from which each transaction in the net sample had been abstracted. Photostat copies were made from each original export declaration and import entry form, and the specific item selected in the sampling circled in ink so as to indicate the particular commodity for which the shipper of the goods was to supply information.

As a result of the commodity selection and sampling technique employed, it appeared feasible to conduct an Origin/Destination Study with a total of only 15,000 observations. These 15,000 observations when expanded to the basic level of full export-import volume of foreign trade indicated a difference of only a few percentage points between tonnages represented by the sample estimate and in actual overseas traffic.

Preparing the Questionnaire. Having completed the sampling process the next step was to prepare a questionnaire to accompany the photostated documents. In cooperation with the Bureau of the Census a questionnaire form for submission to exporters and importers was designed and submitted to the Bureau of the Budget for approval. It contained questions to develop information as to, (a) how the item was acquired for export or how it was disposed of when imported, (b) where acquired or distributed, and (c) by what means it was transported overland. It is noteworthy that of the 15,000 questionnaires in the Great Lakes Overseas Traffic Study which were mailed by the Bureau of the Census, more than 90 percent were completed and returned.

Processing Origin/Destination Data. The overall view, as to the extent of processing necessary in the Origin/Destination study, can best be indicated by stating that at least 12 separate and indentifiable decks of punch cards were necessary to record and correlate data pertaining to all elements of the 15,000 transactions.

The data received in reply to each questionnaire was transferred to punch cards and matched item by item with the original Census foreign-trade cards so as to provide the primary origin/destination data. These data, in addition to Census reference items, include, for each specific commodity shipped, information regarding type of acquisition or disposition; type of transport (inland); commodity detail; weight strata, ports of lading and discharge; point of origin and destination in the United States; distance between U. S. ports and inland point of origin or destination; and weight in pounds, and dollar valuation, for each transaction sampled.

The foregoing procedures after expansion, provided the basic data for use in the transportation analysis phase of the study by which it was possible to establish an area of potential advantage for shipments via the Connecting Channels and the Seaway and to make estimates of traffic allocations to individual harbors on the Great Lakes.

Integration of Origin/Destination Analysis with Other Phases of the Study. The Origin/Destination study is but one phase of the Great Lakes traffic survey. It provides basic data from which other parts of the general traffic study are derived.

Specific carrier cost and freight rate data are incorporated at this point into these punch cards containing origin/destination data so as to make it possible to calculate transportation differentials for each of eleven basic Census commodity groups of import and export cargo moving over trade routes between the United States and twenty separate foreign areas. In a similar manner data are developed with respect to costs and rates for overland transportation, all of which is explained in the transportation analysis section of this paper.

III. Base Period Phase: 1955-1956

Having determined by means of the Origin/Destination study the various points in the United States at which exports and imports enter foreign trade, it is in order to use such information to measure potential traffic expected to be generated in the Great Lakes tributary area.

So far, the boundaries of the Great Lakes area have been defined rather broadly. How to define those limits more exactly and how to estimate the potential traffic of that area is the purpose of the sections of this paper which follow.

Selection of the Base Year. The first step in such a procedure requires that a base year be selected from which to project estimates into the future. For that purpose the two year period 1955-1956 was selected as a representative base.

The methodology involved in this portion of the Great Lakes traffic study is similar to the methodology that would be applied to a research problem in any number of fields. Beginning with the collection and organization of known facts or data, a framework of hypotheses or assumptions is established. From this base, the study proceeds toward the solutions of the basic problems of projection posed by the study.

In an economic study, the findings are not susceptible to "trial runs." The validity of the findings and conclusions of economic analysis must always await the passage of time for proof. However, the validity of the study depends a great deal upon the general acceptance of the known facts or data that stand up

under the "rule of reason." It is the collection and organization of these factual data that comprise the base period phase of this traffic study.

What then constitutes the body of fact upon which this study rests? As already shown, the central problem of the study was the lack of official statistical information regarding origin or destination in the United States of commodities in United States foreign trade especially those likely to be affected by the deep draft ocean traffic that would be brought to the Great Lakes via the St. Lawrence Seaway. Thus, the base period can be used as a bench mark against which this future foreign trade flow of commodities into and out of the Great Lakes tributary region can be measured—commodity flows which need to be identified regarding four primary relationships:

a. Selection of the significant commodity flows to be studied;

b. Identification of the origins of the exports and the destinations of the imports within the Great Lakes region;

c. Determination of the relationship between the export commodities and the producing sectors of the national economy, and between imports and the areas of the United States in which consumed; and

d. Determination of the routings by which the commodities presently move, overland and by sea in United States foreign trade.

Selection of Commodities. In any study it is necessary to establish a system of priorities with regard to the number and detail in which the units are to be analyzed. One form of priority is to determine the relative importance of these commodities basing the selection of each on:

a. Volume and significance of the commodity movement into or out of the Great Lakes area, in terms of both overall tonnage and of specific commodity groupings;

b. Importance of the commodity to the economy of the Great Lakes area and to potential movement via the Connecting Channels and the St. Lawrence Seaway.

c. Level of detail at which the analysis can properly be made, e.g., steel rolling mill products as a whole, or separately for iron and steel bars, ingots, billets, etc., and

d. Adequacy of coverage achieved by the commodity selection in terms of total volumes in the Great Lakes triburary area and Seaway potential as well as with respect to interests of individual Great Lakes ports.

The selection of commodities finally included in a study of this nature requires a detailed analysis of several different tabulations of waterborne foreign commerce, covering a span of years. For the purpose of this report, the two year period 1955-1956 appears to be a representative base. In total some 60 export and 60 import commodity classifications were selected for special attention. In terms of tonnage these items averaged 88 million short tons annually in 1955 and 1956, or almost 90 percent of the volume of commodities to be covered in this traffic study. As mentioned previously, special bulk commodities, such as iron ore, limestone, coal, grains, certain minerals and bulk petroleum, which traditionally have accounted for almost all of the lake commerce, have been treated in special traffic studies.

Origins, Destinations and Routings. After the priorities for study have been established in terms of commodities, the next step is identification of these commodities in terms of their movements into and out of the Great Lakes region. First, the volume of the traffic flow for each commodity is studied as to

specific origins of exports from the Great Lakes region and specific destination of imports into the area. Of importance also was the identification of the foreign areas for which exports from the Great Lakes were destined and the overseas area from which imports into the Lakes region originated. In addition, the particular route taken by the exported or imported commodity is considered in establishing the pattern of its movement through East coast, Gulf or West Coast ports of the United States. Thus, in this phase, the relative importance and significance of each commodity flow is established.

Information regarding commodity flows can be developed from several sources. The primary benefit is, of course, derived from the Census Origin/Destination study which, as already pointed out, provides for the first time on a nation-wide scale significant data regarding the commodity traffic flows mentioned above. For those commodities with overseas traffic potentialities but which are not included in the Census Origin/Destination study it is necessary to analyze foreign trade data statistics, domestic distribution of the commodities from ICC waybill records, together with production and consumption data from Census and other sources.

Producing/Consuming Relationships. Statistics alone do not tell the whole story as to which sector of the nation consumes the imports or produces the exports. Analysis of these relationships can be developed from several sources:

a. A field interview program provides the basic core of information on important aspects of the base period analysis in terms of commodity flows. Although confined to potential shippers within and near the Great Lakes port cities, for the most part, the breadth of coverage in terms of industry provides a representative cross section of commodity flows and the relationship of the commodities to basic producing/consuming sectors of the economy.

b. Staff analysis provides much of the basic data for relating foreign trade products to specific producing/consuming interests by comparing several different statistical codes, such as the Standard Industrial Classification, the Department of Commerce export and import commodity classification and the Association of American Railroads commodity classification which is used by the Interstate Commerce Commission for rail freight movements.

c. Contributions to the methodology of economic research in commodity traffic flows can also be derived from the use of Census data on manufactures, Department of Agriculture publications and the Bureau of Mines reports.

d. A special study of selected minerals, not included in the Census Origin/Destination study, developed data which appears to be highly significant in terms of potential overseas traffic to the Great Lakes area.

Thus, the base period phase sets the stage for the study by selecting the commodities, describing their movements into and out of the Great Lakes region, and identifying them in terms of domestic geographic areas of origin or destination within the economic sectors of production and consumption of the United States.

Finally, in terms of volume, movements to and from specific foreign areas are identified by the patterns of traffic by land and by sea. The identification of the points of origin and destination and the routes between the domestic points and the foreign areas provides the basic framework for a transportation cost and/or rate analysis which will be discussed later.

IV. Projection Phase: 1960-2010

Having once established the identity and volume of tonnage moving to or from the Great Lakes area during the base period (1955-1956) it is necessary to take a long-range look at the prospects of commerce as a whole and the selected commodities in particular. This projection for the period 1960 to 2010, is required in order to compare estimates of benefits and costs on an average annual basis for the assumed economic life of the project which is usually set at 50 years.

Thus, in regard to overseas commerce of the Great Lakes, it becomes necessary to make estimates in what is perhaps one of the most variable elements of the National economy—the prospective long-range level of general cargo in foreign trade. Stated in another way, it is a question, of determining whether tonnage that contributes during the early years of the project to establish a favorable benefit cost ratio can reasonably be expected to continue for the 50-year assumed economic life of the project. Conversely it may also be a question of determining whether tonnage that does not contribute to an adequate extent in the early years of the project, may build up to a sufficient level during the life of the project.

National, Great Lakes Area, and Port Traffic Potentials. In making projections of future traffic a subjective approach is suggested, with analysis proceeding from the general to the particular. In all instances such projections will proceed first from analyses conducted at a national level, then to analyses at the "Great Lakes Area" level, and finally to analyses at local port levels.

Basically, the projections take into account such indicators as trends in Gross National Product, industrial production, and population to establish, insofar as data are possible, a reasonable indication of basic growth rates in the economies of the principal countries, or geographic areas accounting for the bulk of our foreign commerce. Using these national indicators as a bench mark, projections can be made at a national level of trade for the 11 major commodity groups for which transportation differential data are being developed.

The projection estimates for these 11 commodities are based upon analyses of the conditions of trade applicable to principal, or representative, items in each of the eleven commodity groups. Inasmuch as there are some 400 commodities involved for exports and imports combined, only the most significant items of trade should be chosen for study.

For each of the commodities selected, a separate analysis is made as to historical trends in national production, long-range potential trends in the total of imports and exports, and historical relationships between the level of foreign trade and domestic production. Where available data permits, analysis also covers shifting levels of consumption and indicated changes between competing and/or substitute products.

The resulting projections, by commodity group represent estimates properly weighted to account for the relative importance of commodities in each category. These commodity group projections are used as a bench mark in making the estimates of the long run potential for foreign trade at a regional level.

It is to be emphasized that such estimates are not forecasts in the true sense of the word. Rather, after evaluating presently available data, and taking into account the indicated shifting composition of such trade, they should be considered as conservative levels of trade which can be expected in the future.

Estimates Used. In recognition that foreign trade is subject to substantial fluctuation from year to year, estimates of potential traffic will show high and low ranges between which the trade levels are likely to vary. Inasmuch as conservative estimates are used, the actual future level of trade may well be considerably higher.

Mention should be made also of commercial policy with regard to prospective levels of foreign trade since the question invariably arises, as to whether any forecast of trade can adequately take into account the possible imposition of mechanical devices to restrict trade for balance of payments, for protection or for other reasons. In this respect, for the purpose of deriving conservative minimum estimates of future trade levels, it can be assumed that there will be no increase in the historical relationships between exports and domestic production, and imports and levels of domestic consumption. Thus, forecasts will be based upon growth prospects only and not provide for any increase in the share of the market commanded by foreign goods.

Moving from the national to the regional or Great Lakes area level, the national projections are evaluated on the basis of the relative importance of the producing/consuming sector of the regional economy, as indicated by regional growth trends. This phase of the analysis also involves use of the regional economic data collected for this purpose by the Corps from many industries through interviews.

At a port level, the estimates of potential traffic represent an evaluation of information collected through staff research, which in turn is evaluated through first-hand discussion with industry officials actually engaged in foreign trade and through information developed in cooperation with local communities at public hearings.

Admittedly, in the light of the present status of the art of forecasting, projections are subject to a degree of error, but by taking what is considered a conservative position it is believed that a sound projection of the trade of the 1960-2010 period can be achieved.

V. Transportation Analysis Phase

After determining the points of origin and destination of foreign trade in the United States, the next step is to establish the limits of the Great Lakes tributary area. This can be done by comparing the transportation expenditures for Great Lakes-Seaway routings and for coastal port routings and establishing a "differential" or "transportation advantage." The magnitude of this transportation advantage can be termed the "driving force" which would orient traffic toward the Great Lakes ports or toward the seaboard, assuming that factors relating to shipping service, terminal facilities and such are equal at the alternative ports. The problem then is to develop a method of measuring this differential or relative transportation advantage.

The determination of transportation differentials requires at least 30,000 calculations, or two for each of the 15,000 observations in the sample. Maximum simplification of these calculations can be achieved through the use of algebraic formulas, punch cards and tabulating machines.

It is recognized that transportation differentials can be developed on the basis of both (a) freight rates and (b) carrier costs. From the viewpoint of methodology, the treatment of rates or costs is very similar. For the purpose of illustration this paper explains the technique developed on the basis of carrier cost data.

The transportation cost for overseas export or import shipments is composed of four segments: (a) ocean transportation costs, (b) overland transportation costs, (c) tolls, and (d) cargo handling charges. In view of the long-term nature of the determinations being made, handling charges for the future period were assumed to be equalized for alternative routings. The level of tolls used are those recently announced by the United States and Canada. This leaves only the ocean and inland transportation costs to be determined.

Ocean Transportation Costs. The ocean carrier's cost is based upon the vessel's operating, administrative and capital expenses for a round voyage of a C-2 type ship on specified routes. This total route cost is translated into the cost per cubic foot of the vessel's utilized cargo space. By application of a factor representing the number of cubic feet of vessel cargo space occupied by a hundredweight of a specific commodity, the cost of transporting the hundredweight of that commodity can be determined. In other words, the ocean transportation cost per hundredweight (cwt.) for a specific commodity to or from a designated foreign area is equal to the vessel cost per cubic foot of cargo space multiplied by a density stowage factor which represents the number of cubic feet per hundredweight of that commodity. Or to express it in a formula:

$$\text{Ocean cost per cwt.} = S = (\text{no. cu. ft. per cwt.}) \times (\text{vessel cost per cu. ft.})$$

$$\underline{\text{or}} \quad S = \frac{(\text{no. of cu. ft. per gross ton})}{(\text{no. of cwt. per gross ton})} \times (\text{vessel cost per cu. ft.})$$

$$S = \frac{(\text{stowage factor})}{(2240 \text{ lb}/100 \text{ lb})} \times (\text{vessel cost per cu. ft.})$$

$$S = \frac{F}{22.4} \times k$$

where S = Ocean transportation cost per cwt. for a specified commodity to a specified foreign area.

F = Stowage factor for the specified commodity (no. of cubic feet of commodity per 2240 lb. of commodity).[1]

k = One-way vessel cost per cubic foot of cargo space for a specified routing. It is based upon the total vessel cost for specified routing divided by number of cubic feet of space utilized for cargo. Toll charges against the vessel are included.

To establish formulas for the ocean transportation cost via a coastal routing compared to a Seaway routing, algebraic symbols are provided for designated items. With symbols associated with the Great Lakes-Seaway routings distinguished from ocean transport via a seaboard port by a prime accent mark ('), the ocean transportation costs are:

(a) Via a coastal port $= S = \dfrac{F}{22.4} \times k$

(b) Via the Great Lakes-Seaway $= S' = \dfrac{F}{22.4} \times k'$

1. References which contain computed stowage factors include: Modern Ship Stowage by Joseph Leeming, U.S. Department of Commerce, and The Stowage Red Book compiled and edited by Harry R. Hanlin, Walter W. Weller and Charles J. Fagg and published by the Traffic Publishing Company, Inc., New York, 1944.

To establish the cost per cubic foot of vessel cargo space (k), the following formulas are developed:

$$\text{Cost per cu. ft.} = k = \frac{V}{n} = \frac{\text{vessel cost for designated route}}{\text{no. of cu. ft. of cargo space}}$$

where $v = d_s v_s + d_p v_p + Z$

and d_s = no. of days at sea

v_s = vessel cost per day at sea

d_p = no. of days in port

v_p = vessel cost per day in port

Z = other costs and charges related to the vessel, such as tolls on the vessel, pilotage, wharfage, etc.

n = no. of cu. ft. of vessel cargo space utilized on the specified route.

therefore $$k = \frac{V}{n} = \frac{d_s v_s + d_p v_p + Z}{n}$$

if k = vessel cost per cu. ft. for shipment via a coastal port

k' = vessel cost per cu. ft. for shipment via the Great Lakes-Seaway

then $$k = \frac{V}{n} = \frac{d_s v_s + d_p v_p + Z}{n}$$

$$k' = \frac{V'}{n} = \frac{d'_s v_s + d'_p v_p + Z'}{n}$$

and $$S = \frac{F}{22.4n}\,(d_s v_s + d_p v_p + Z)$$

$$S' = \frac{F}{22.4n}\,(d'_s v_s + d'_p v_p + Z')$$

Overland Transportation Cost. For the purpose of the determination of transportation cost differentials, the overland transportation costs can be represented by railroad costs. In the Census Origin/Destination study, it was found that the railroads transported the greatest proportion of the shipments. The railroad cost formulas used are those developed and published by the Interstate Commerce Commission. They represent costs based on 1956 operations with adjustments to reflect wage and price levels as of January 1, 1958.[1]

The rail costs are composed of terminal costs plus line-haul costs and can be expressed as follows:

1. Interstate Commerce Commission, Rail Carload Cost Scales by Territories as of January 1, 1958, Statement No. 2-58, Washington, D. C. The Cost Finding Section, Bureau of Accounts, Cost Finding and Valuation, March 1958.

Rail cost = (Terminal cost) + (line-haul cost)[1]

$$\underline{\text{or}}\ R = \left(\frac{T}{c} + t\right) + X\left(\frac{H}{c} + h\right)$$

<u>where</u> R = Rail costs per cwt. of commodity
T = Per carload part of the terminal cost (varies with type of car)
c = Number of cwt. per carload (varies with commodity)
t = Per cwt. part of the terminal cost
X = Distance of haul
H = Per car-mile part of the line-haul cost (varies with type of car)
h = Per cwt.-mile part of the line-haul cost

<u>then</u> rail cost to coastal port is $R = \left(\frac{T}{c} + t\right) + X\left(\frac{H}{c} + h\right)$

<u>and</u> rail cost to the Great Lakes port is $R' = \left(\frac{T}{c} + t\right) + X'\left(\frac{H}{c} + h\right)$

<u>where</u> X = distance from inland point to seaboard port
X' = distance from inland point to Great Lakes port

As part of the method of utilizing the most economical routing of a shipment, barge costs and truck costs are examined and compared to the rail cost. The determination of carrier costs for the barge shipments are based upon a method similar to that used in the ocean cost determination. Truck costs are examined to determine if such costs yield a more economical routing for certain shipments. The truck costs are based upon data supplied by the Cost Finding Section of the Interstate Commerce Commission.

As part of the problem of handling the great mass of data, the method used to obtain the inland distance of the 30,000 shipments to the actual port of exit or entry is designed for machine computation. Each geographical point in the study (a port or an inland city) is located on a mile grid. The straight line distance between points is determined by means of coordinates on the mileage grid. Then straight line distances are expanded to short line rail distances which in turn are adjusted for circuity to obtain an approximation of actual distance.

<u>Transportation Cost Differentials</u>. In determining the transportation cost differentials, the question of cargo handling costs becomes a problem in determining the <u>difference</u> in cargo handling costs for any two routes being compared. Since the foreign terminus of a particular alternate route is the same, and since the costs for handling the same cargo at the Great Lakes port and U. S. coastal port, should be, over the long run, essentially the same, such handling costs cancel out and are therefore omitted as a factor in determination of the transportation cost differential. In addition to cargo handling costs, certain tolls are assessed against the ship's cargo when the routing is via the Seaway, the Suez Canal, or the Panama Canal.

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1. The rail costs, whether measured either by out-of-pocket costs (long-run variable costs) or by fully distributed costs (long-run fixed and variable costs) can be calculated by this formula. To establish comparability between the factors included in the rail costs and the ocean costs, adjustments can be made in the cost data. As noted in a following section, the final determination of the transportation differential.

To express the toll charges in algebraic terms, the following symbols are used:

Let L = tolls for routings via U. S. Coastal ports

L' = tolls for routings via Great Lakes ports

Summary. The total transportation cost for each routing is the sum of the overland, ocean and tolls segments.

If Y = total transportation cost via a coastal port

and Y' = total transportation cost via the Great Lakes-St. Lawrence

then $Y = S + R + L$ (ship cost + rail cost + tolls)

and $Y' = S' + R' + L'$ (ship cost + rail cost + tolls)

The differential, "D" is:

$$D = Y - Y'$$

or

$$D = (S + R + L) - (S' + R' + L')$$

(If D is positive (+) the cost advantage is via the Great Lakes; if negative (-) the cost advantage is via the seaboard).

If the expressions for the components of the total cost are substituted in the above formula and the expression is simplified by assuming that the number of days in port is the same by either routing ($d_p = d'_p$), the differential can be reduced to the following expression:

$$D = \frac{F}{22.4n}\left(v_s(d_s - d'_s) + (Z - Z')\right) + (X - X')\left(\frac{H}{c} + h\right) + (L - L')$$

These formulas will establish the values of the differential "D" for each of the 15,000 transactions in the Census Origin/Destination study. These determinations will show the volume of the Base Period (1955-1956) traffic with a transportation cost advantage via the Great Lakes-Seaway. These data will also be utilized to construct "differential contour" patterns (lines of the same advantage) which can be applied to the distribution of the projected foreign trade origins and destinations. The differential contour pattern will indicate the specified "driving force" (such as 25 cents per cwt., 50 cents per cwt., or other amount) that is applied to a particular shipment to indicate the transportation advantage for that shipment via the Seaway. It will be possible thus to show the volume of foreign trade within specific categories of transportation advantage for a routing via the Great Lakes-St. Lawrence Seaway and the Connecting Channels. Conversely, advantages via seaboard ports will also be apparent.

The ocean distance, travel time and carrier cost between Great Lakes ports and overseas ports from the Great Lakes area is somewhat greater than for alternate routes through most seaboard ports. This difference is reflected in existing ocean freight rates. When considered in conjunction with the transportation differentials discussed above it becomes more apparent that the advantages of the Great Lakes-St. Lawrence route may be largely the difference in the overland haul. In general freight rates and carrier costs show that the greatest transportation advantage via the Seaway is for shipments originating or terminating in or near Great Lakes ports. The next advantage zone includes those cities at a substantially greater distance from a coastal port than from

a Great Lakes port. The least advantage is shown where points of origin or destination are about the same distance from coastal port and Great Lakes port.

VI. Potential Traffic Phase: 1960-2010

The preceding sections of this paper have shown how methodology is developed to obtain basic data regarding:

1. The magnitude of foreign trade in the base year period (1955-1956) and its projection into the 1960-2010 period;
2. Where these exports and imports comprising this foreign overseas trade originate and terminate in the United States; and
3. What it costs to transport this overseas cargo by land and by sea between points of origin and destination.

The foregoing factors are brought together in the potential traffic phase so that the volume of overseas traffic generated in the Great Lakes Tributary area is analyzed in terms of the probable volume that may actually transit the Seaway. This estimate of total potential Seaway traffic moving via United States Great Lakes ports is based largely on the findings of the analysis of the most economical routings either through coastal ports or by Great Lakes ports. The traffic estimate is refined by considering more subjective factors as the Seaway Season and other institutional factors affecting commodity movements.

Finally, the volume of potential Seaway traffic must be further delineated by estimating the volume of traffic at individual harbors on the Great Lakes. The methodology applied in allocating the total potential Seaway traffic to individual Great Lakes Harbors represents in miniature the determination of the Seaway traffic volume for the Great Lakes area as a whole. That is, the tributary area for each port is delimited in terms of the traffic generated within the port's natural area of influence; the volume that can most economically move through the particular port is determined; and finally the port's potential traffic is estimated. Again, the final estimate of potential Seaway traffic is refined in terms of more subjective considerations which include the local condition at each port in terms of facilities and institutional forces which affect commodity movements.

In many cases it is impossible to determine through which port a commodity will move, since it is probable that any of two and perhaps three ports might be used at different times. Thus, port traffic estimates must also contain an element of sound judgment above and beyond the mechanical methodology by which a particular estimate is derived.

SUMMARY AND CONCLUSIONS

In summary then, the objective has been to obtain information as comprehensive in scope and as factual as possible for use in estimating traffic potentials to evaluate requirements for harbor improvements.

As has been seen, the methodology moves through several successive stages of analysis at each of which the data are subjected to a greater degree of refinement.

The initial analysis is directed primarily at the national level and efforts concentrated on identifying the important volume items in total United States

overseas trade. Analysis is then narrowed to the Great Lakes tributary area and attention focused on the share of the total national overseas traffic generated by this region.

Consideration is next given to the probable future level of trade in commodities significant to the Great Lakes region. Estimates of such traffic are then tested for the most economical routing. From analyses of data collected in field interviews and from staff research, an evaluation is then made of other factors likely to influence the use of the Great Lakes. From a synthesis of all these data an informed estimate as to the total potential tonnage that may move via the Great Lakes, the Connecting Channels and the St. Lawrence system becomes possible. Following the determination of the Great Lakes-overseas traffic potential, an estimate can be made of the traffic expected to move through the individual lake ports. This estimate is based upon consideration of a variety of factors, such as transportation cost differentials, the magnitude of the economic hinterland of individual ports, and shippers' opinions as to possible future routings of the more important items of commerce.

The sources of information used in an analysis of this nature are varied and extensive. Not only is a wealth of Government statistics available as basic background data, but many Government agencies have been most cooperative in supplying unpublished information.

The Department of the Interior, for example, furnished information on the present and prospective flow of minerals in the Great Lakes area; the Department of Agriculture supplied valuable information on many agricultural products; and the Interstate Commerce Commission supplied published information on overland carrier costs.

Of particular interest, because it proved to be an important key to basic problems of the Great Lakes-overseas traffic survey, has been the opportunity to analyze on a nation-wide basis the origins and destinations in the United States of foreign trade cargo. Through this Origin/Destination study it has been possible to develop much transportation information heretofore not available. However, as pointed out in the Introduction, it is desired to restate that any formulas or methods proposed in connection therewith, but not yet accepted by the U. S. Army Corps of Engineers, reflect only the views of the author and not necessarily those of the Corps.

The interest of the Corps of Engineers in traffic studies such as the present one is to obtain a comprehensive view of traffic potentials for the purpose of evaluating the "economic justification" of proposed harbor projects.

At this time, note should be taken of some of the problems that have been encountered in the study. For example, the lack of a continuing "foreign trade census" prevents development of historical perspectives as to the regional importance and trends of international trade and commerce. The rather extensive data available as to the pattern of freight movements by the railroads is not complemented by comparable data regarding freight movements by truck.

Far from being put forth as the final word in methodology for determining overseas traffic potentials, this paper is offered with a view to inducing others to explore further the possibilities presented here.

ACKNOWLEDGEMENTS

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NATURAL BY-PASSING OF SAND AT COASTAL INLETS

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ABSTRACT

At coastal inlets two main principles in by-passing of littoral drift sand by natural action occur: by-passing on offshore bar and by-passing by tidal flow action. This article discusses these two ways of by-passing as a function of littoral drift and tidal flow characteristics of the inlet in question.

INTRODUCTION

By-passing, in this article, is to be understood in the way that the material, after a short interruption caused by an inlet, pass, channel, jetty or other kind of "littoral barrier", is given back to the normal littoral drift zone a short distance downdrift from the littoral barrier.

If nature did not by-pass sand across inlets, passes, and channels on sea shores a number of "marine forelands" including barriers, spits and entire peninsulas would not exist. A typical example of this is found in Florida which was built up of sand washed down by rivers and streams from the Appalachian Highland and carried southward, crossing estuaries and tidal inlets, for final deposition in the huge barrier and ridge systems which we call Florida.

The two main principles in by-passing of sand by natural action are:

By-passing on an offshore bar, and

By-passing by tidal flow action

Most cases present a combination of these two methods.

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a. Presented at the October, 1958 ASCE Convention in New York, N. Y.

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A submerged bar in front of an inlet or harbor entrance on a littoral drift coast will often function as a "bridge" upon which sand material is carried across the inlet or entrance. Every channel dredged through the bar will, therefore, be subject to deposits.

By-passing by tidal flow action takes place when littoral deposits by flood currents are spoiled out of the inlet again by ebb currents in the downdrift direction. Both bar and tidal flow by-passing include cases with irregular transfer of large amounts of materials in migrating sand humps or by change in the location of channels.

One can distinguish between inlets or entrances with predominant bar by-passing and inlets which may present a predominant tidal flow by-passing by considering the ratio $\frac{M_{mean}}{Q_{max}} = r$ between the magnitude of littoral drift (M_{mean} in cu. yd. per year) and the quantity of flow through the inlet (Q_{max} in cu. yd. per sec. under spring tide conditions).

If this ratio is high, bar by-passing is predominant; a low ratio indicates that conditions favoring predominant tidal flow by-passing exist. Meanwhile whether or not such by-passing actually takes place depends on whether or not it is possible to use the tidal flow for transferring material in the down-drift direction. This depends, among other things, upon the inlet configuration. Inlets exist which, due to strong tidal currents, are detrimental to any transfer of material because ebb-currents jet the material so far out into the ocean that it is lost forever for the shore.

Table I mentioned under "Discussion" gives values of M_{mean}, Q_{max} and r for various inlets including those described in the following paragraphs. It is shown that inlets with $r < 10 - 20$ have a predominant tidal flow by-passing while inlets with $r > 200 - 300$ have predominant bar by-passing.

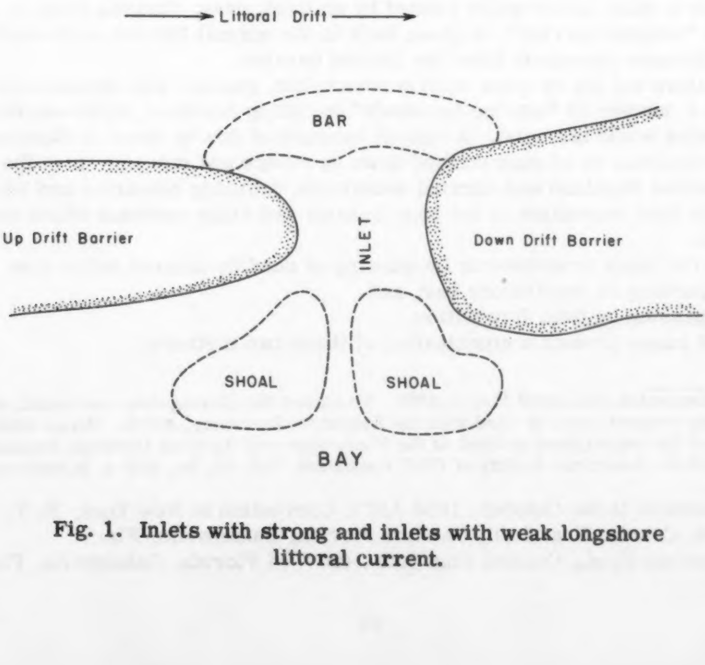


Fig. 1. Inlets with strong and inlets with weak longshore littoral current.

1. Bar By-Passing—Limited Tidal Action

1.1 The Principle Involved

Fig. 1 shows a barrier with an inlet. Littoral drift material passes along the barrier. At the downdrift end it continues on its way across the inlet on a submerged bar, the extent and depth of which depends on the amount and character of the material which by-passes and the intensity of wave and current action. By increasing amounts of littoral materials the bar area increases and depths decrease. Increase in wave action results in a smaller and more streamlined bar with greater depth. Stronger longshore currents also result in a narrower and more streamlined bar with perhaps one predominant channel, while weaker longshore currents may result in more and shallower channels. The mechanics of bar by-passing may be a more or less continuous process or it may partly take place in greater irregular sand waves or humps which migrate across the inlet on the bar.

From the abovementioned, it is clear that if satisfactory natural by-passing is to be established the criterion must be that the longshore drift capacity be kept on the same level, regardless of the existence of the inlet. Knowing the distribution of littoral drift and longshore currents at different depths in the normal beach and bottom profile, it is possible to estimate the

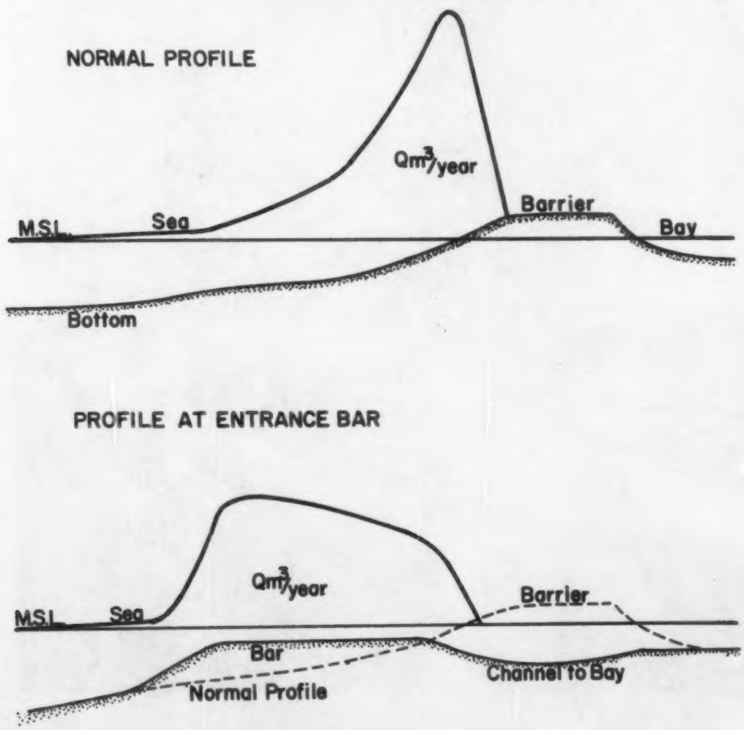


Fig. 2. The principle involved in by-passing of sand on a submerged bar.

necessary depth for a certain width or the necessary width for a certain depth of the bar, both for a certain wave action and longshore current action (Fig. 2). Meanwhile, wave and current action vary and this will cause irregularities in the by-passing. Experience demonstrates that such by-passing cannot exist without considerable wave action and, moreover, that the depth over the bar is usually limited to the breaker depth for normal and predominant storm waves. Width of the bar is as mentioned above mainly a function of the velocity of the longshore current.

It is interesting to compare the mechanics of bar by-passing with the natural by-passing around the California promontories as described by Parker Trask in (14). The active by-passing zone seems here to extend to 30 ft. depth, and even deeper sand is transferred by the waves stirring up material to be moved by currents.

It is obvious that man has few chances to arrange satisfactory navigation and by-passing conditions simultaneously because breaking waves necessary for by-passing are hazardous to navigation. This is the reason why inlets with bar by-passing are usually only useful for small crafts. Navigation improvements have only limited interest and often are very difficult to justify and finance.

Fig. 3. Matanzas Inlet, Florida.

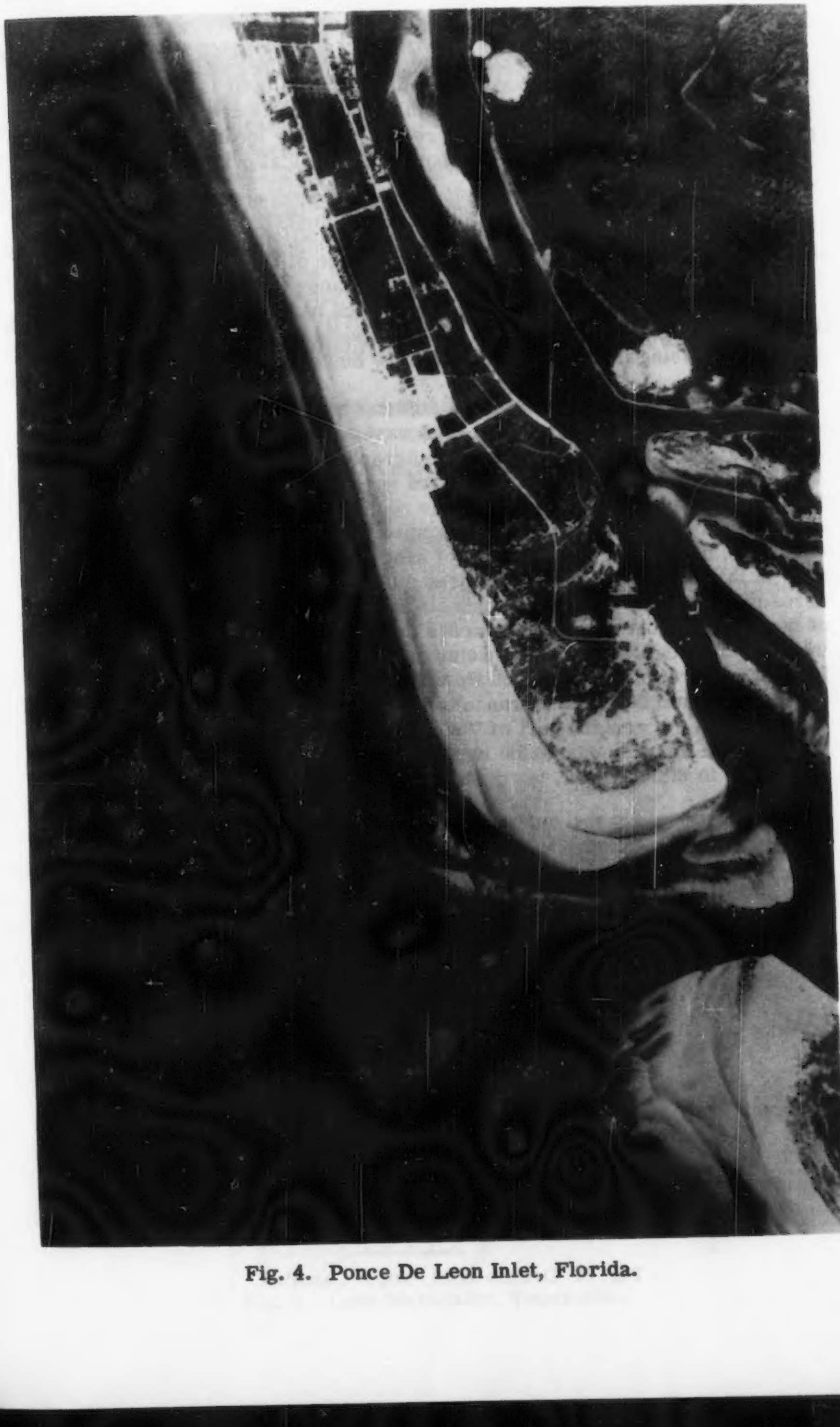


Fig. 4. Ponce De Leon Inlet, Florida.

1.2 Examples of Inlets with Bar By-Passing

Unimproved Conditions.—Fig. 3 and Fig. 4 show Matanzas and Ponce de Leon Inlets on the Florida east coast where littoral drift is about 400,000 to 500,000 cu. yd./year to the south. Both of these inlets are very old and have probably been almost stationary where they are now located for hundreds of years. The normal tidal range is about 3 ft. but tidal prisms are rather small in both cases.

Matanzas Inlet with 3 to 5 ft. depth on the bar is an example of very irregular bar by-passing. A shifting channel sometimes gets too close to the downdrift beach thereby causing serious erosion of this beach (1955 - 1958). The inlet is not very useful for navigation because of its ocean side shoals.

Ponce de Leon Inlet has a half-moon shaped bar with depth 8 - 10 ft. This bar is very unstable because of shifting channels and transfer of sand is very irregular causing irregular erosion of New Smyrna Beach on the downdrift side.

Oregon Inlet in North Carolina is depicted on English maps from the 16th century. Tidal range is about 5 ft. and wave action heavy with 10 - 15 ft. storm waves. Fig. 5 shows aerial photographs from 1945 and 1949. During this period the inlet migrated southward several hundred feet by extension of the northern barrier and erosion of the southern. At the same time the channel across the outer half-moon shaped bar with 8 - 10 ft. depth shifted northward by which a great amount of sand was transferred across the inlet. Sand in this case is by-passed in three different ways: in suspension directly across the inlet, by shifting of channels, and indirectly by erosion of the tip of the downdrift barrier which benefits the downdrift beaches. Since 1846 the inlet has migrated 1.5 - 2 miles southward but in the 1952 to 1958 period 2,500 ft. was cut off the northern tip widening the inlet considerably.

Another example is the entrance to the 18,000 sq. kilometer Lake Maracaibo in the western part of Venezuela (Fig. 6). A detailed description of the region of Lake Maracaibo is given in (12). When oil started moving out of the lake to the open sea, the draft of the vessels was limited to 9 ft. by an

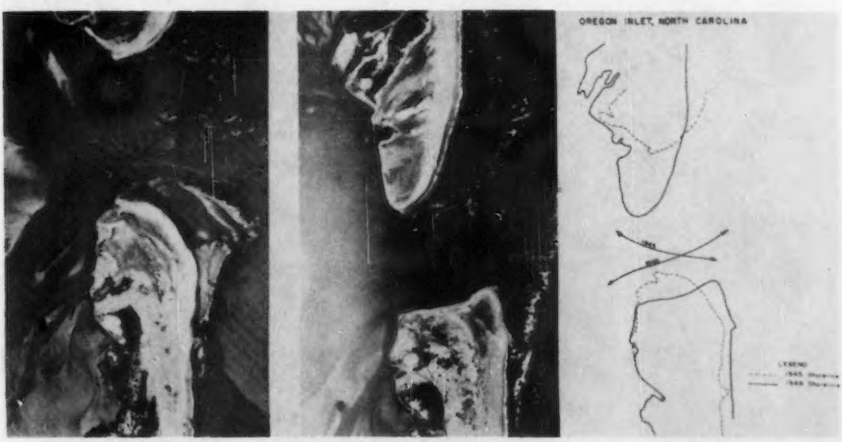


Fig. 5. Oregon Inlet, North Carolina.

outer and inner bar. Wave action is heavy on the 10 - 14 ft. deep outer bar and the westward littoral drift is very strong, while the normal tidal range is only about 1 ft. Surveys show that the channel over the outer bar migrated westward up to 3 ft. per day and that a new channel would break through east of the migrating deteriorating channel. The cycle of migration consumed a period of about twenty years. In 1953 the Venezuelan government initiated a project to provide a 35 ft. channel. This project was completed recently but considerable maintenance dredging is expected because of the deep channel.

In Portugal the inlet at Figueira Da Foz in the estuary of the River Mondego (Fig. 7) is an example of an inlet where natural bar by-passing has created satisfactory conditions for small craft for a very long period of time.

Littoral drift along the Portuguese west coast is chiefly from north to south and tidal range is about 10 ft. The inlet is being studied by the Hydraulics Division of the "Laboratorio Nacional". The scope of the study as described in (10) is:

(a) to investigate drifting conditions in the area of Cape Mondego and the adjacent beaches, particularly

  (1) to determine whether sand movement occurs or not from north to south beyond Cape Mondego;
  (2) to get an idea of the comparative intensity of drift in the foreshore and middle depth area;
  (3) to study the drifting pattern along Buarcos Bay; and to estimate the speed with which sand coming from the sandy coast north of Cape Mondego reaches the mouth of River Mondego.

(b) to determine the conditions of passage of littoral drift material beyond the river mouth in a north-south direction.

Fig. 6. Lake Maracaibo, Venezuela.

The solutions of these problems were of great interest among other reasons for verification of the accuracy with which natural conditions have been reproduced in the movable-bed model built to study jetty improvement of the River Mondego estuary and Figueira Da Foz harbor.

As the area in question had a surface of some tens of square miles, and as there was no knowledge of the rate of littoral drift and the time required for the tracer material dumped into the sea to reach the harbor, it was impossible to use a short-life radioactive isotope. The requirement had to be at least "90 days" with maximum radioactivity. Four tons of irradiated AG 110 was chosen for the experiments, the result of which, as shown in Fig. 7 (figures indicating counts per minute), was:

(a) there is a considerable drift along Cape Mondego.
(b) in the area of Cape Mondego only drift in the foreshore could be detected, as the difficulty of sampling (or measuring) prevented drift observations in the middle-depth zones.
(c) it was verified that littoral drift material passed across the mouth of the River Mondego, but it was impossible to determine how this passage took place.

<u>Improved Conditions</u>.—"Improved conditions" include inlets which have been improved for navigation. Improvements must consider such installations as jetties, groins, dams, sluices or similar structures which may help nature establish a better transfer arrangement without dredging operations.

A small but interesting case has been in operation at Thorsminde (Thors Inlet) on the Danish North Sea Coast. Fig. 8a and Fig. 8b are charts of the inlet which has navigation locks as well as locks for flow. Tidal range is about 1 ft. and wave action mostly heavy with 5 - 12 ft. waves. Littoral drift is estimated to be at least 500,000 cu. yd./year. The inlet is protected by two jetties. Until 1944 both were 500 ft. long. During the period 1942- 1947 two groins were built on the updrift side and the north jetty was extended 200 ft. The groins should catch excessive amounts of material migrating

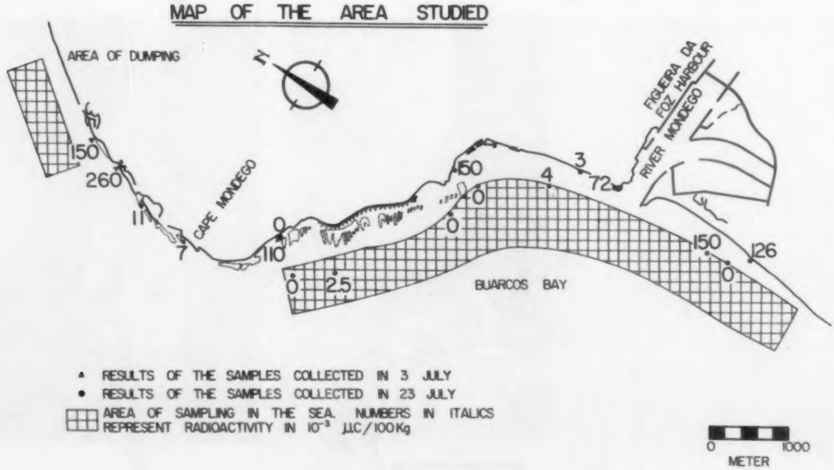


Fig. 7. Figueira da Foz, Portugal.

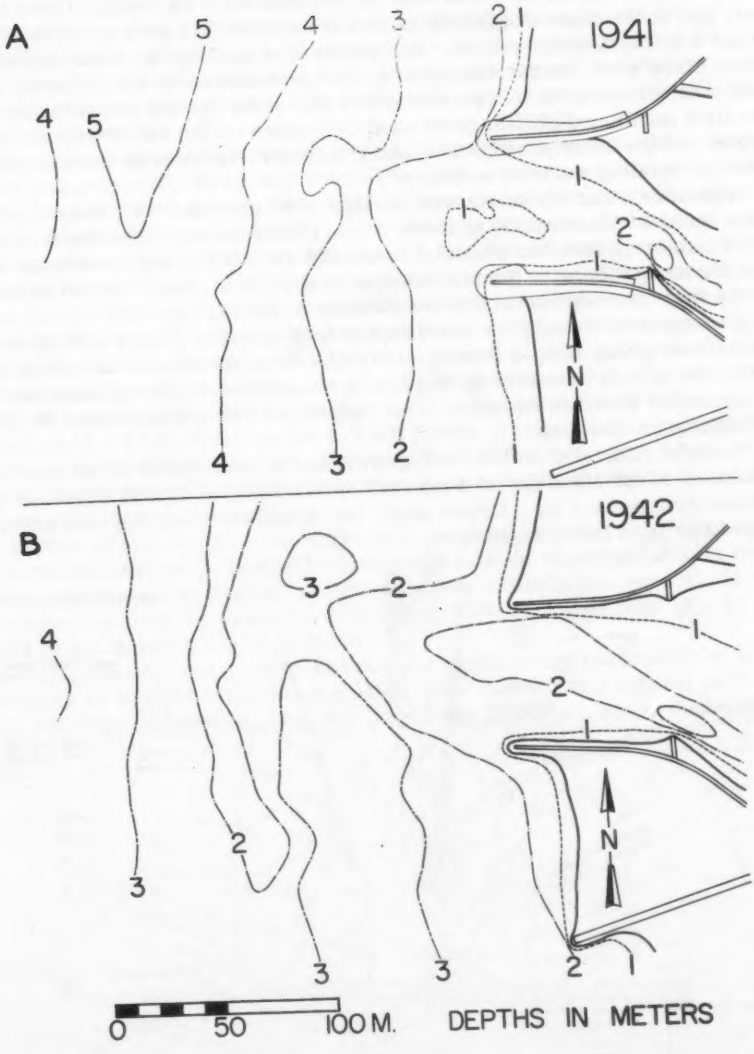


Fig. 8. Thorsminde Inlet, Denmark.

toward the inlet and the jetty-extension should push material to be by-passed farther seaward. Fig. 8a shows the situation on June 23, 1941, when the inlet conditions were particularly bad, with depths less than 3 ft. between the jetties (normally 6 - 7 ft.). It can be seen that there is no bar in front of the inlet. Fig. 8b shows the conditions on November 7 - 11, 1942. There is a bar, and at the same time the inlet conditions are very good—with depth of about 8 ft. between the jetties. The problem of sanding up, which always takes place when the bar disappears, can be explained by the different distribution of littoral drift in a profile with a bar and a profile without a bar. At the first mentioned profile much sand by-passes on the bar where waves break. At the other profile most material migrates close to the shoreline, thereby shoaling the inlet entrance.

A similar situation exists with another inlet provided with sluices and located south of Thorsminde at Hvide Sande (White Sands). The depth on the bar which by-passes the material is usually 10 - 12 ft., and conditions are fair for navigation with fishing boats up to about 6 ft. draft. Model experiments were carried out on this installation in the 1920's.

It is apparent from these two cases that by-passing occurs satisfactorily and without giving rise to leeside erosion, with a bar of a certain depth and width, but in both cases irregularities in the amount of littoral drift may momentarily result in shoaling of the respective inlets which must be cleared by flushing or dredging.

The Inlet of Aveiro on the Portugese Atlantic coast north of the earlier mentioned River Mondego as described by Abecasis in (1) and (2) shows a similar development on a larger scale, but tidal flow resulting in a half-moon shaped bar is of more importance.

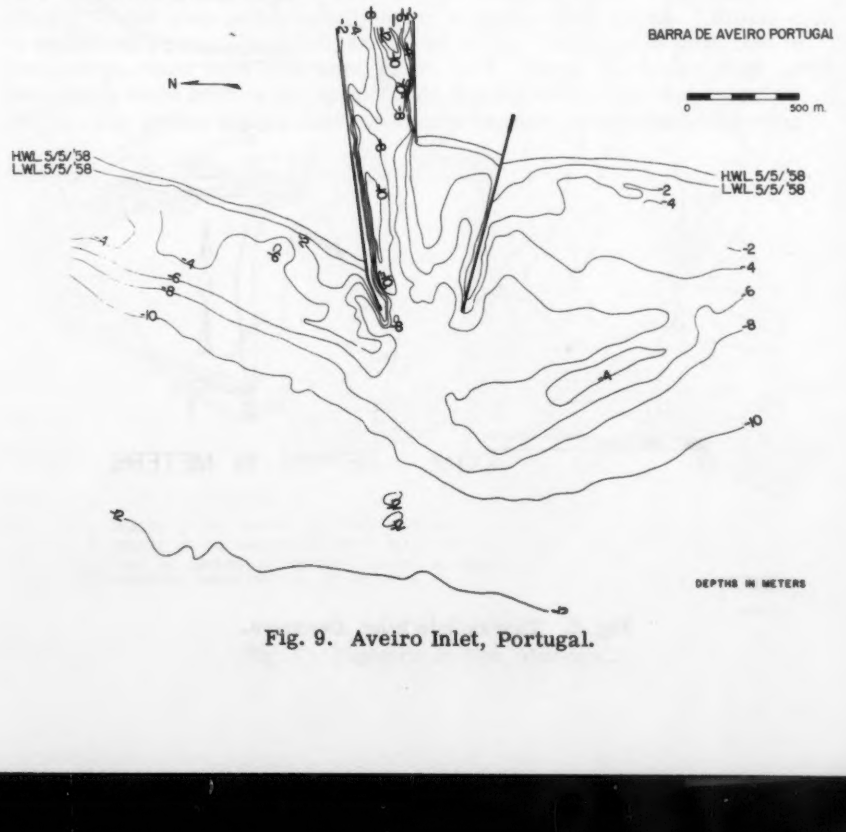


Fig. 9. Aveiro Inlet, Portugal.

The history of the inlet dates from the 10th century and shows a number of migrating as well as stable periods. Attempts to stabilize the inlet started in the 17th century but only one project, an artifically cut inlet in 1808 by Carvalho, gave a long-range result. During the following period of about 140 years, sand migrated on a bar across the inlet in a predominantly southward direction; during years of normal weather conditions the entrance being merely fixed by a jetty built along the southern bank extending to the low water line. Navigation in the inlet was considerable even if the configuration of the shoreline at the entrance was changeable and the original project was not well maintained.

After some time, to meet the increased navigation requirements, a new project was prepared, which was started in 1949 and is now practically completed. Detailed surveys as mentioned by Abecasis in (2) clearly demonstrate that littoral sands are not retained by the inlet on their way down-coast because the volume of sand expelled by the ebb tide considerably exceeds that brought in by the flood. "And if so, the project being carried on correctly, solved the problem of assuring the depths required (indeed, more than required) in the channel with the least interference with the littoral drift regimen: in fact a strictly localized interference, both in time and in the space, as is also confirmed by the absence of any permanent erosion effects on the downdrift section of the sea shoreline." (cit. Abecasis in (2) p. 411). Apparently a 12 - 15 ft. shoal on the downdrift side is important in the transfer of sand to the downdrift beaches when combined with heavy wave action. Meanwhile the present inlet situation is still so new that it may be too early to draw very definite conclusions.

The Fort Pierce Inlet on the lower east coast of Florida (Fig. 10) should be mentioned here even though tidal currents play an important role in its natural transfer arrangement. Meanwhile, it is unlikely that sand would be transferred without the existence of a rather wide rock reef with 10 - 12 ft. depths on the downdrift side of the inlet.

Investigations carried out by the Coastal Engineering Laboratory of the University of Florida and published in (5) show that out of a littoral drift of about 200,000 - 250,000 cu. yd./year only about 20,000 cu. yd. is deposited on

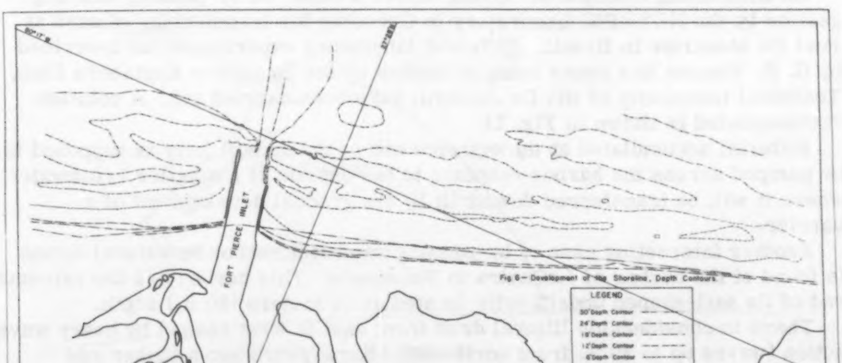


Fig. 10. Ft. Pierce Inlet, Florida.

the updrift side—while about 160,000 to 200,000 cu. yd. pass through or over the 2000 ft. long updrift jetty. While approximately 40,000 cu. yd. of this quantity is dredged or deposited on bay shoals, 120,000 to 160,000 cu. yd. are jetted out in the ocean again by the very strong ebb currents (5 - 6 ft./sec.). Part of the material jetted out, perhaps 40,000 cu. yd., is lost to deep water; about 20,000 cu. yd. washed back into the inlet channel through the south jetty; and about 100,000 to 150,000 cu. yd. evidently given back to the downdrift side but apparently not until some few miles farther downdrift. Leeside erosion in the amount of about 100,000 cu. yd./year occurs in the first two to four miles south of the inlet according to the 1958 survey.

How this by-passing mechanism works is not known but there is evidence that the flat rock reef with depths 10 - 12 ft. commencing at the extreme end of the 1200 ft. long south jetty and extending downdrift plays an important role in this transfer action. The Laboratory has suggested that this question be further studied by radioactive tracing.

1.3 By-Passing at Harbors

A special way of bar by-passing occurs at harbors on littoral drift coasts with weak or no tidal currents in their entrances.

An example may be found on the Danish North Sea coast at Hirtshals Harbor. A heavy littoral drift, perhaps 500,000 - 1,000,000 cu. yd./year, comes from the west with a strong longshore current. Part of the drifting sand is deposited in "tongues" along the updrift jetty while a great part passes the extended updrift jetty for depositing by a large clockwise eddy current in a shoal on the downdrift side. This shoal is gradually growing larger by deposits ranging between 50,000 and 200,000 cu. yd. a year. Maintenance dredging is necessary in the 7 - 8 meter (25 ft.) deep entrance channel to the harbor. The development in recent years shows decreasing depths on the shoal (10 - 13 ft.). At the same time the shoal has extended farther downdrift with the result that the leeside shore is now being nourished from the shoal thanks to heavy wave action which apparently brings some of the material back to the shore.

An interesting example in "giving nature a hand" on by-passing is a suggestion by the NEYRPIC Laboratory in Grenoble for transferring of sand at Port De Mucuripe in Brazil. Different laboratory experiments as described by G. E. Vincent in a paper being published by the Brazilean Engineers Club, Technical University of Rio De Janeiro, have been carried out. A solution recommended is shown in Fig. 11.

Material accumulated at the extreme end of the updrift jetty is supposed to be pumped across the harbor entrance to the outside of a smaller breakwater where it will be transferred downdrift by the gradual development of a barrier.

Another interesting case of by-passing sand at a harbor by natural action is found at the harbor of LaGuaira in Venezuela. This harbor has the extreme end of its nail-shaped updrift jetty located at 18 meters (60 ft.) depth.

There is considerable littoral drift from east to west caused by heavy wave action (waves up to 20 ft. from northeast). Some years ago a tanker ran aground midway out on the jetty at 30 - 40 ft. depth and accumulated, in a short time, a great amount of sand behind it, demonstrating the heavy drift. Meanwhile, there has so far been no accumulation at the extreme end of the jetty and it is believed that the great depth may be responsible for this.

Reference is, in this respect, made to Cornaglia's theory (Italy). Based on the experience with erosion of steep shores and gently sloping shores, Cornaglia claims that a neutral "line" or "depth" exists for any condition of wave action. Inside this depth material moves mainly onshore; and outside, mainly away from shore thanks to the influence of gravity.

Cornaglia's theory is not generally accepted but some model experiments at the Massachusetts Institute of Technology Hydrodynamics Laboratory(7) seem to indicate that the theory may be valid under certain circumstances such as also demonstrated by nature.

Fig. 12 shows project for breakwaters for the improvement of the fishing port of Scheveningen on the Dutch coast. The alignment of the jetties has been determined from model experiments intending to secure a maximum of sand by-passing of material and a minimum of depositing in the harbor entrance.

2. By-Passing by Tidal Flow Action

2.1 The Principles Involved

Where the ratio between the net littoral drift and the tidal flow $\frac{M_{mean}}{Q_{max}} = r$ is relatively small the tidal flow usually plays an important part in successful by-passing of material.

Unimproved Inlets.—In general sand transfer by tidal flow takes place in two different ways, namely by

(a) migration of channels and bars, and by
(b) transport of sand by tidal flow in the channels.

Re (a).—Tidal channels in inlets (particularly those running between the gorge and the ocean) are subject to migration. This means that they change location continuously, moving from one side of the inlet to the other. In

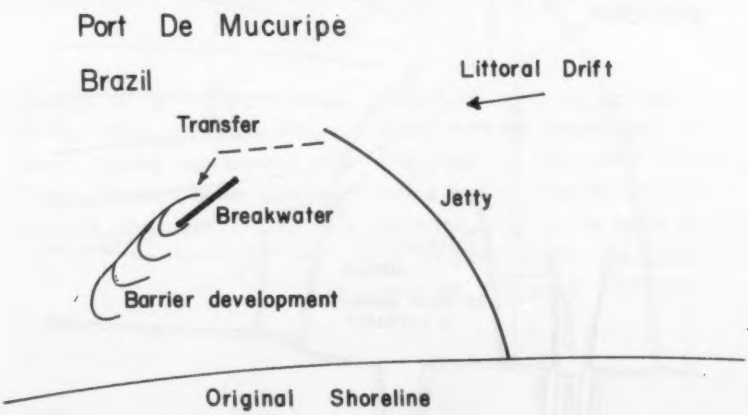


Fig. 11. Port De Mucuripe, Brazil.

Fig. 13 this principle is demonstrated by phases 1 and 2 of a tidal channel system. Channels in phase 1 are numbered I, II, III, and IV. In phase 2 the locations of these channels have changed compared to phase 1 and a new channel "O" has developed. In this example the channels move from left to right and bars or shoals between the channels move in the same direction with the result that a bar occasionally joins the downdrift side beach where the sand is distributed along the downdrift coast by waves and littoral currents.

In most cases, migration of tidal channels takes place in the direction of the littoral drift. Sand is transported over the bar under the influence of waves and deposited on the updrift bank of the channels, thus forcing the shifting.

Because of the fact that tidal channels behave in the same way as channels in regular rivers, in curved sections sand motion along the bottom will have a component towards the inner curve. This, in turn, will increase the rate of migration of the channel in the direction of the downdrift side curve.

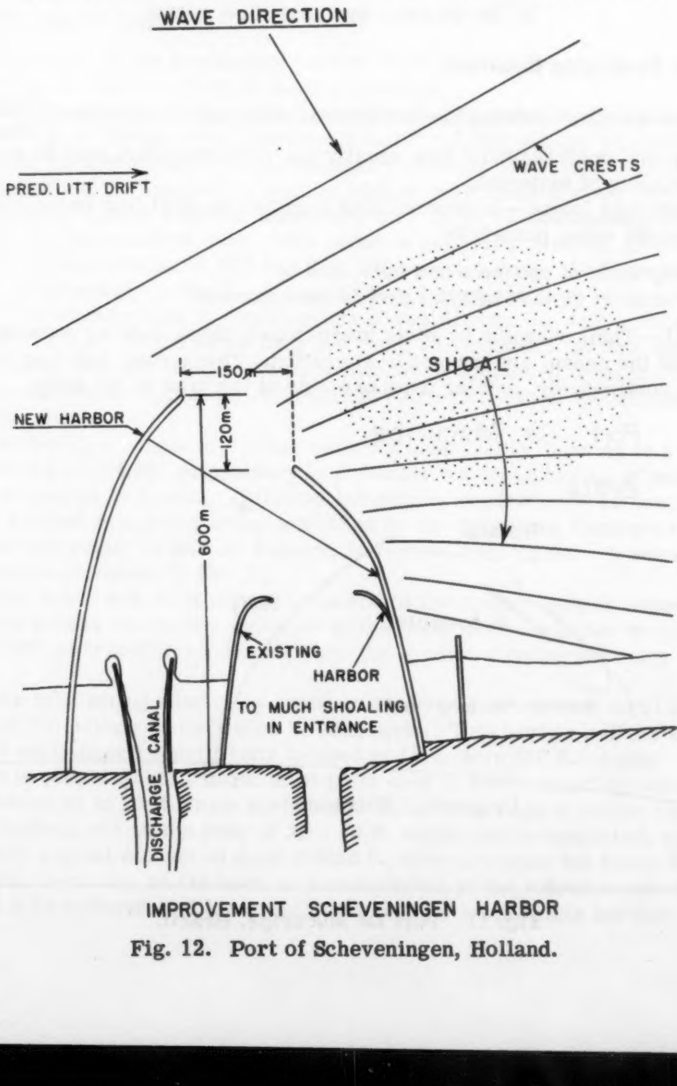
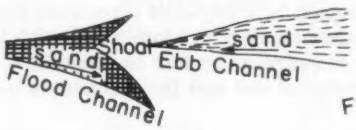
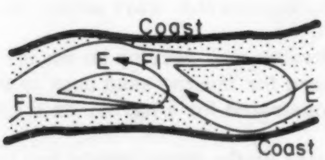


Fig. 12. Port of Scheveningen, Holland.

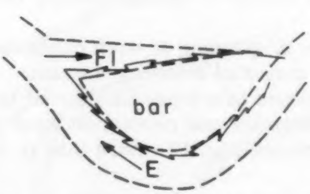


Sketch of the mutual evasion of flood-and ebb-channels by means of a forked tongue.

Sketch of a mutual evasion with flank attack of flood-and ebb-channels.



Sketch of so-called circulating sand currents, the sand moving up-stream in flood-channel, down-stream in ebb-channel.



Sketch of the true up- and down-stream movement of the sand in so-called circulating sand currents. A grain of sand may come back to its original place; dredging may be of small avail.

Fig. 13. Ebb and Flood Channel Characteristics.

Re (b).—In the vicinity of tidal inlets, the (generally) strong tidal currents in the inlet change the littoral drift pattern entirely. Along the uninterrupted coastline wave action is generally the predominant cause for the transportation of material. In the vicinity of tidal inlets, however, transport of material takes place under the combined effect of waves and tidal currents.

In tidal rivers, estuaries and inlets tidal channels can usually be identified as either flood or ebb channels. Flood channels carry predominantly flood flow, causing a resultant sand transport in a bayward direction; they usually have a shoal at the end. Ebb channels carry predominantly ebb flow and have a resultant material transport seaward and a bar or shoal at the end. Compare Fig. 13 where different types of ebb and flood channels have been indicated (ref. 15).

It is remarkable that ebb and flood channels never tend to be connected, but always leave a bar between them over which sand migrates. Fig. 14 shows schematically an ebb and flood channel system in an inlet area. Vectors indicate the resultant direction of sand transport in different channels. Sand particles make a zig-zag movement through the inlet from the updrift side to the downdrift side.

Jetty-Improved Inlets.—During the first phase of a jetty improvement a great amount of sand is generally accumulated along the updrift beach. With an increasing amount of accumulation, by-passing of material gradually develops.

Fig. 15 shows three jetty-improved tidal inlets. In each case the ebb current has enough cleaning capacity to maintain the inlet channel by flushing action.

Adequate by-passing in such cases needs considerable wave action to move the flushed material shoreward again.

The presence of a (special shaped) breakwater may develop an advantageous diffraction and refraction wave pattern for moving sand back to the beach. Compare Fig. 12 where this is demonstrated for a harbor on a littoral drift coast.

At tidal inlets material is often accumulated on the inner shoals and this decreases the amount of material available for by-passing. The inner shoal will usually not contribute much to the by-passing mechanism because normally it is located outside the area of appreciable wave action and strong tidal currents. It sometimes provides a convenient source for artificial by-passing by pumping of material from the shoal to the downdrift side.

2.2 Examples of Inlets with By-Passing by Tidal Flow Action

Unimproved Inlets.—An example of predominant tidal flow by-passing is the two groups of estuaries which form the southern and northern part of the Dutch coast.

Fig. 16 shows the configuration of the Dutch coastline in general. Predominant wave action from the southwest causes a predominant northward littoral drift although heavy storms infrequently may create high waves from the northwest.

Movement of sand is greatly influenced by the existence of comparatively strong tidal currents parallel to the coast.

The flood current runs in a northward direction and is the stronger. Its maximum velocity reaches 4 ft./sec. at approximately 1 mile from the shore. The southward ebb current is weaker. Its maximum velocity is about 3 ft./sec.

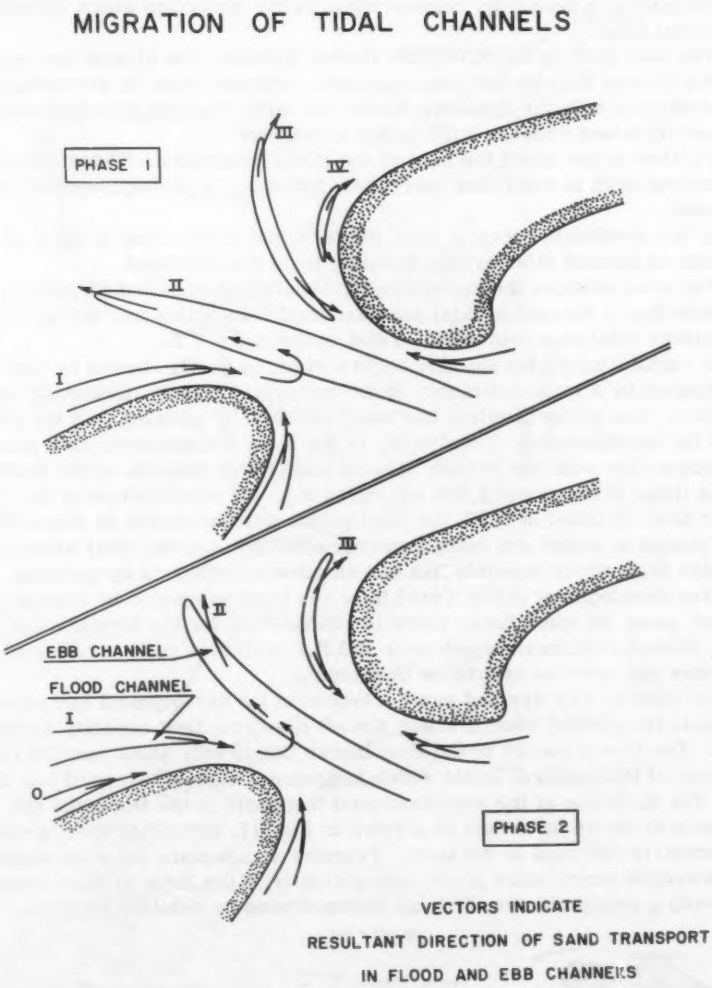


Fig. 14. Migration of tidal channels.

The southernmost inlet, the Wester Schelde, forms part of the seaway to the port of Antwerp in Belgium. Its tidal prism is about 1600 sq. miles ft. and the cross-section near the entrance about 865,000 $ft.^2$ The mean tidal range in this area is 10 - 12 ft.

Large quantities of sand are by-passed across this wide estuary by the tidal flow. However, the downdrift shore is insufficiently nourished, primarily because of a deep tidal channel close to the shoreline which carries the material away.

The next inlet to the north, the Ooster Schelde, has almost the same size as the Wester Schelde but tidal currents, although equal in strength, are less concentrated with the apparent result that sand transfer is better and the downdrift island coast is sufficiently nourished.

Farther to the north the size of the inlets decreases. Therefore, the ratio of littoral drift to tidal flow increases, resulting in an improvement of by-passing.

At the northwest group of tidal inlets on the Dutch coast a chain of barrier islands is located at a varying distance from the mainland.

The area between the islands and the mainland is called "Wadden". The Wadden Sea is formed of tidal sand and mud flats which are dry at low water. Numerous tidal channels exist. Tidal range is 5 - 7 ft.

In various localities erosion occurs which is partly caused by tidal channel shifting or by a local deficiency in the material balance. Generally speaking, however, this group of inlets has adequate sand by-passing. At the present time the southernmost, Texel Inlet, is the least satisfactory. The size of it is comparable with the Wester Schelde and Ooster Schelde of the southwest group (tidal prism about 1,600 sq. miles ft.). By construction of the Usselmeer dam (finished in 1932) the tidal prism was increased by about 20% (due to a change in phase and reflection characteristics of the tidal wave in the Wadden Sea), which possibly has had an adverse effect on by-passing.

Free development of the Texel Inlet has been hampered by a heavy protection along its south bank, which is responsible for the formation of a very deep channel (maximum depth over 150 ft.), with high concentration of currents and adverse effects on by-passing.

The inlet of Vlie has had more freedom in its development and presents adequate by-passing even if it has almost the same flow capacity as the Texel Inlet. The flow capacity of the Eyerlandse Gat is only about one-fifth of the capacity of the adjacent inlets which indicates a higher degree of bar transfer. The direction of the resultant sand transport in the flood and ebb channels is shown by means of arrows in Fig. 17, indicating the zig-zag movement of the sand in the inlet. Transfer is adequate but nourishment of the downdrift shore takes place intermittently in the form of sand waves with decreasing amplitude downdrift as demonstrated by detailed surveys.

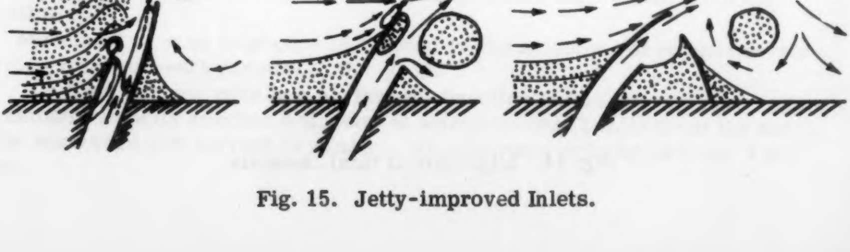


Fig. 15. Jetty-improved Inlets.

Fig. 16. Dutch Coast Line.

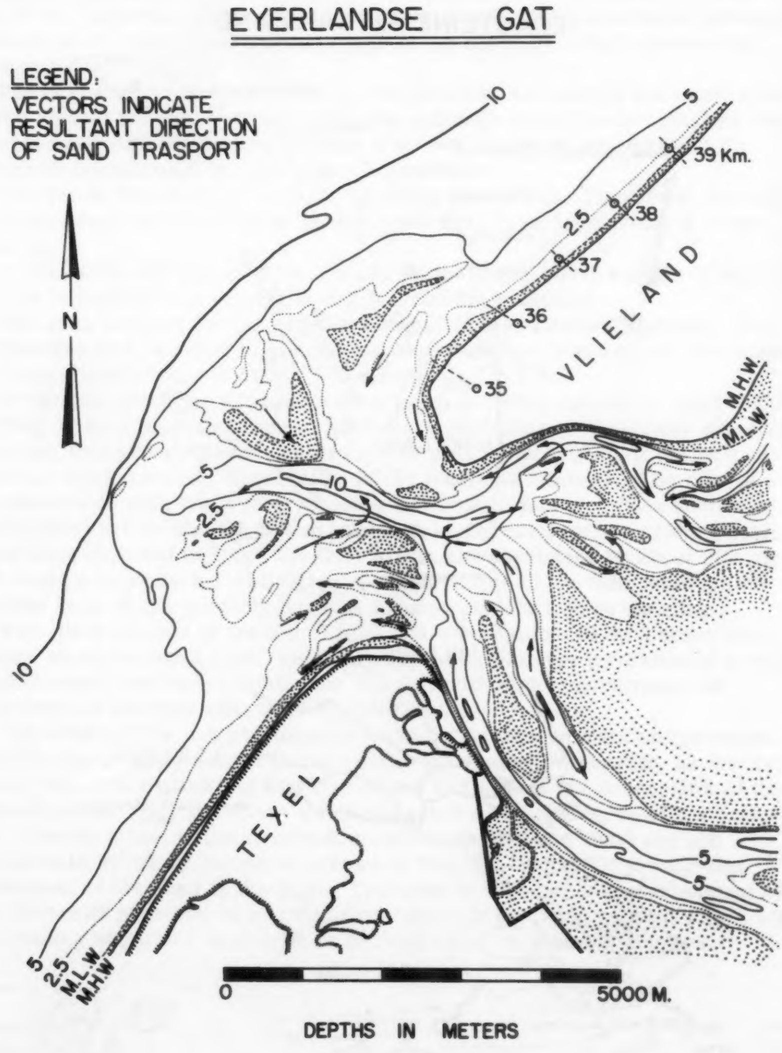


Fig. 17. Resultant sand transport vectors in "Eyerlandse Gat".

In locations where the predominant wave direction makes only a small angle with the shoreline, refraction of waves over the shoals may lead to a reversed direction of the littoral drift immediately downdrift of the inlet, and prevent adequate by-passing of sand in this area. Some inlets on the West Coast of Florida as e.g., the Sarasota Pass, present this problem.(4)

The estuary of the river Humber (Fig. 18) separating Yorkshire from Lincolnshire in East England (in a 15 ft. tidal range area) seems to present a very unique case of sand transfer by tidal currents. The littoral drift on the Holderness coast is southward and material accumulates at Spurn Head. Comparison of old maps indicates that Spurn Head in periods extended itself up to 170 ft. per year southward. In other periods the rate of advance was only 15 ft. per year. Several times Spurn Head was cut off from Yorkshire. As mentioned by Steers in (13) Spurn Head undoubtedly does not represent more than a very small part of the littoral drift accumulation from the Holderness coast. On the other side of the Humber in Lincolnshire there are large masses of sand at Donna Nook. These are probably not formed by the

Fig. 18. Humber River Entrance, England.

erosion of the Lincolnshire coast because there the native rocks are mostly warp and clay, without stones.

It is unlikely that anything coarser than sand would cross the Humber estuary but the fact that Spurn Head has at times become detached from Yorkshire may explain the situation because, first detached from the mainland, the island may have worked its way across the river. Meanwhile there appears to be no record of this process in action.

Whether a continuous transfer of sand by tidal currents across the estuary may take place still remains to be investigated but the estuary has a rather advantageous configuration for such transfer.

Another example of transfer of sand by combined action of tidal currents and waves is Graadyb on the Danish North Sea the northernmost of the inlets in the Frisian Island group at the Port of Esbjerg.

Improved Inlets.—Fig. 19 is harbor entrance at Abidjan, Ivory Coast, Africa. A cut was made to connect the ocean with a lagoon to accommodate vessels of 27 ft. draft. Sand coming from the west is deposited by the flood current at "M"; the ebb current, which is strongly concentrated at that point, transports it in the direction of "P", where part of it settles in a deep hole in the sea bottom.

Fig. 20 shows the entrance of Lagos Harbor (Africa). In order to counteract erosion on the downdrift coast model experiments were carried out, from which the following recommendations were made:

a) construction of groin at "C" to counteract erosion of section "BC"

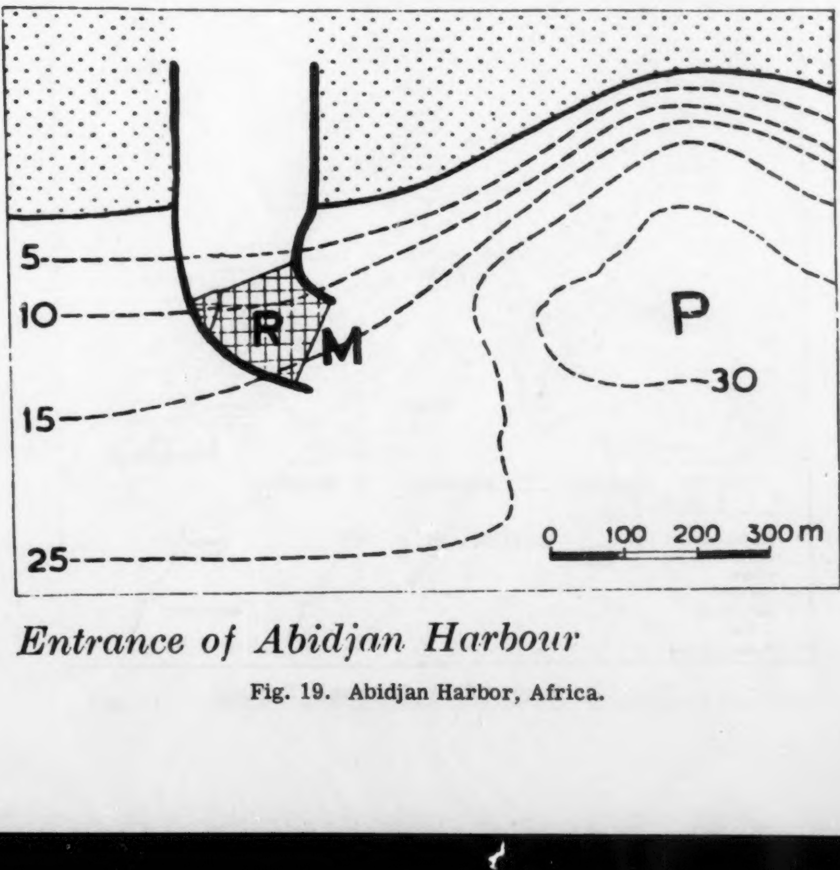


Fig. 19. Abidjan Harbor, Africa.

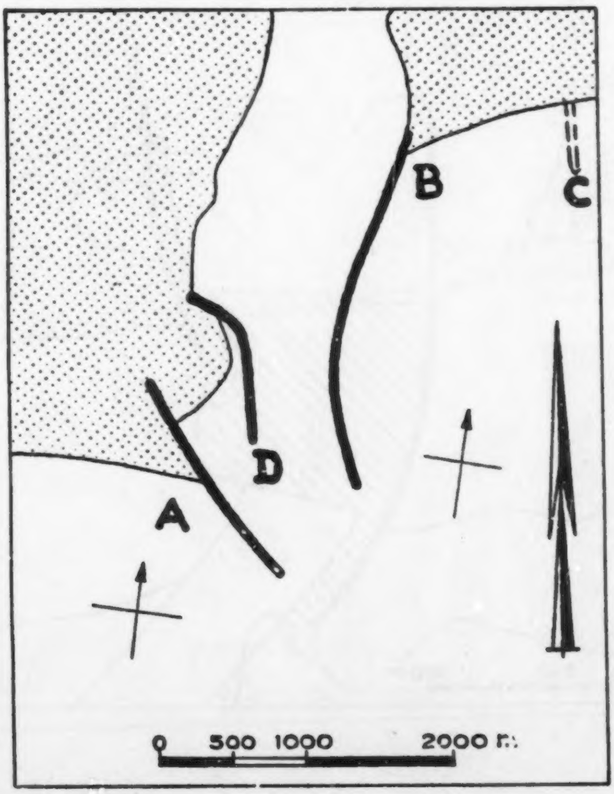
b) realignment of moles, with or without deposit of sand at "D" in the navigable channel. Currents and swell transport this from "D" to "BC"

c) construction of a stationary sand pump at "A" to pump sand from "A" to "B" should also be considered.

Fig. 21 is the mouth of the Volta River on the Gold Coast (Africa). Sand is carried around the end of the west mole by flood currents and deposited at "M". From there it is carried eastward by ebb currents which are concentrated at that point because of the special alignment of the eastern mole. This sand is, for the greater part, deposited in the downdrift area.

The abovementioned harbors at Abidjan, Lagos and the Volta River entrance have all been investigated by hydraulic model studies in the Hydraulic Laboratory at Delft, Holland. In each of these cases, a satisfactory solution for sand by-passing was obtained.(17)

San Francisco Harbor (Fig. 22) has a large tidal prism (2880 sq. miles ft.) flowing through the Golden Gate (875,000 sq. ft.). The strong tidal currents



Entrance of Lagos Harbour

Fig. 20. Lagos Harbor, Africa.

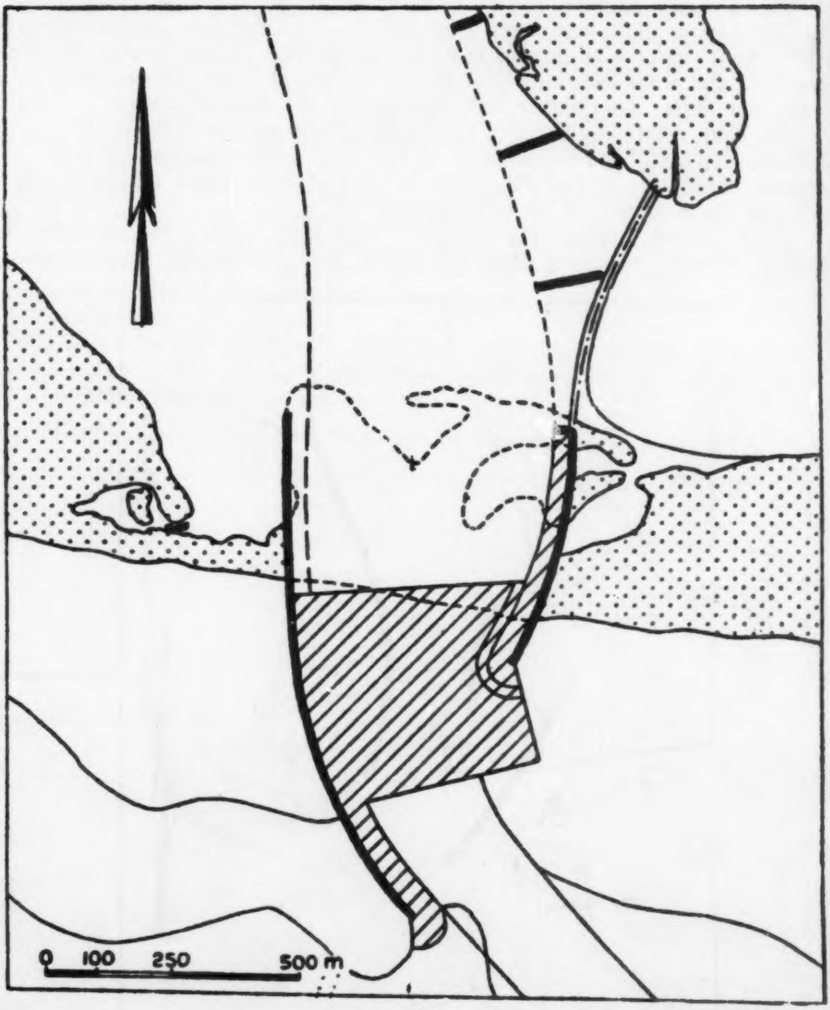


Fig. 21. Volta River Entrance, Africa.

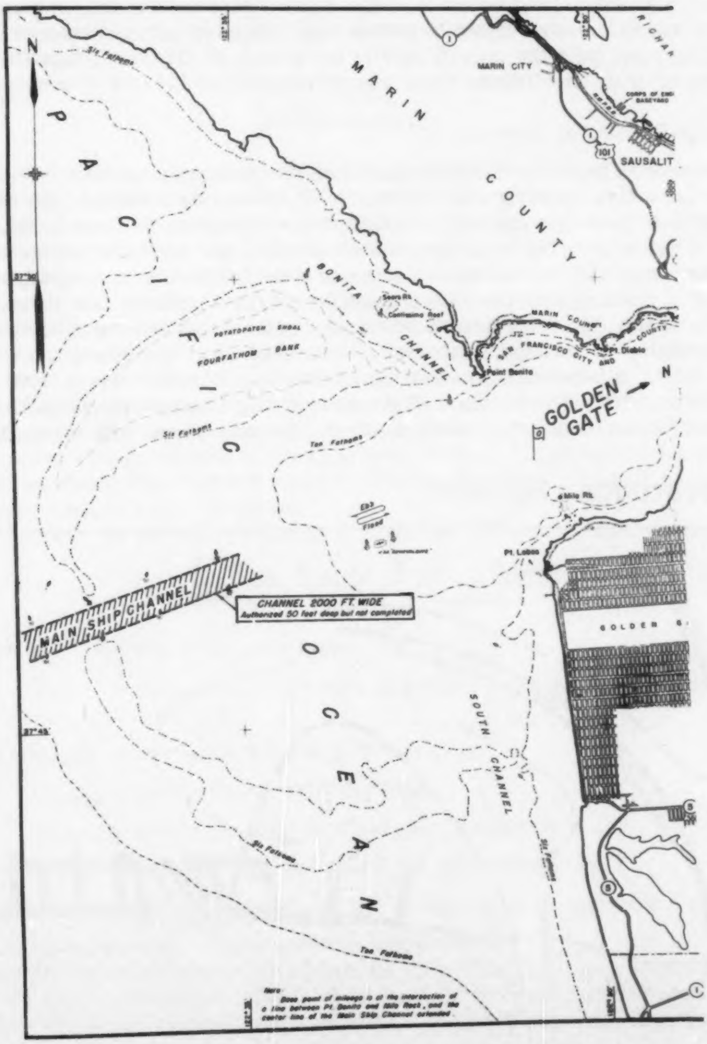


Fig. 22. San Francisco Harbor, California.

and heavy wave action are responsible for the huge half-moon shaped offshore bar with depths from 12 - 18 ft. It is most likely that littoral drift material passes across the 50 ft. deep entrance channel but the process, perhaps a zig-zag movement of sand on the bottom, is not known. Downdrift side erosion obviously does not occur.

The jetty-improved entrance to the river Weichsel, Poland by-passes sand in the way that the river entrance has been designed and constructed for spoiling sand from the updrift side in the downdrift direction, depositing it temporarily on an offshore shoal where wave action carries it onshore again.

2.3 By-Passing at Harbors

Mentioned in this connection should be the Zeebrugge Harbor in Belgium (Fig. 23). This harbor protected by a 4500 ft. long nail-shaped jetty was for a long time greatly bothered by silt deposits amounting to about 5,000,000 cu. yd. per year. The tidal range is about 12 ft. and the tidal currents outside the harbor up to 5 - 6 ft./sec. For some time the harbor was equipped with a 1300 ft. opening (claire-voie) permitting tidal currents to flow through the harbor basin. This was satisfactory. Heavy deposits, mainly silt, continued and dredging of the deposits endangered the economy of the harbor.

In order to improve this situation model experiments were carried out after World War II in Belgium (Waterbouwkundig Laboratorium) and in Holland (Waterloopkundig Laboratorium). An experiment with strong flood

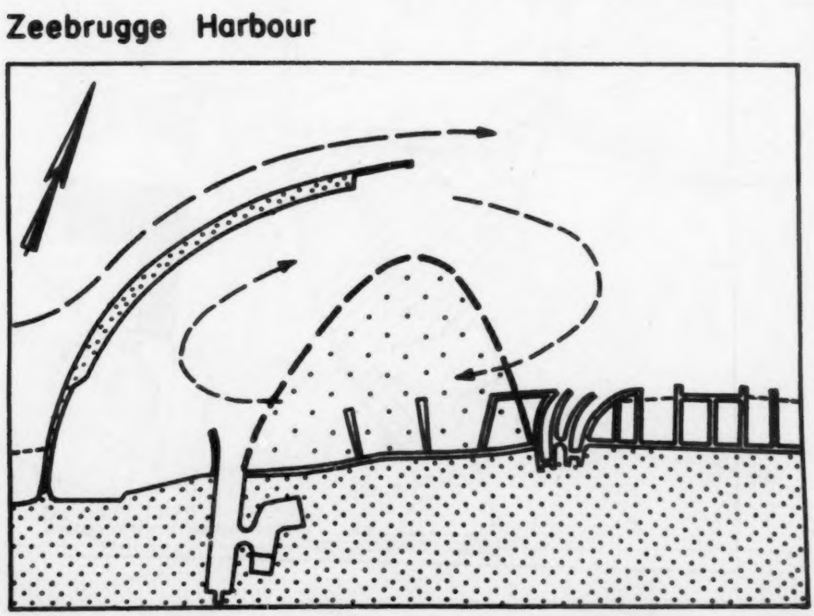


Fig. 23. Zeebruge Harbor, Belgium.

currents is shown in Fig. 23. The result of the construction of the big circular jetty on the shore-side was the elimination of a large silt depositing eddy current in the harbor basin. The amount of silt deposits was reduced to less than 50%. The remainder of the material (mainly silt) by-passes the harbor with the tidal currents.

In model experiments with Karlsruhe harbor in Germany (river harbor) special jetty-configuration secured by-passing of heavy bed-load transport in the river flow.

3. Discussion

From the abovementioned it is clear that natural by-passing depends upon a number of different factors such as littoral drift magnitude including longshore current velocity and wave action, quantity and velocity of tidal flow, material characteristics, etc. These variables are not independent and depend themselves on other parameters as e.g., shoreline and offshore bottom configuration and other geological and geographical factors.

As a first approximation those factors which seem to have a predominant influence on by-passing are considered thereby ignoring those variables whose effect are probably of secondary importance. Although the material characteristics obviously are of importance in the by-passing mechanism, this parameter can be eliminated if considerations are restricted to inlets on sandy coasts with grain size 0.15 to 0.5 mm. Within this group the variation in grain size material does not seem to have predominant effect on the by-passing procedure and mechanism.

Accordingly the by-passing factor P is tentatively written as follows:

$$P = f\left(\frac{E_{wa}}{M_{mean}}, \frac{M_{mean}}{Q_{max}}, \frac{M_{max} - M_{mean}}{M_{mean}}\right)$$

in which P = the by-passing factor

E_{wa} = wave energy of waves of a certain period from a certain direction

M_{max} = maximum littoral drift per year

M_{mean} = mean littoral drift per year

Q_{max} = maximum tidal flow per sec. through the inlet

This equation is inhomogeneous but useful for discussion.

The parameter $\frac{E_{wa}}{M_{mean}}$ depends on the amount of wave energy and on the angle of approach of the waves with the shoreline.

Considering cases where "fair" natural by-passing has been established at inlets as e.g., at Ponce De Leon Inlet in Florida, Oregon Inlet in North Carolina which was recently dredged to 12 - 14 ft., Thorsminde and White Sands in Denmark, and Figueira Da Foz and Aveiro in Portugal, under unimproved as well as improved conditions, it is characteristic that they all represent inlets with moderate to heavy wave action, causing much stir-up of littoral drift material. As a consequence of this littoral drift is strong with considerable suspended load transport utilizing the transport capacity of longshore currents. In all cases by-passing is a rather "continuous operation" without long interruptions.

With reference to Fig. 2 it can, therefore, be said that bar by-passing is possible whenever wave action and longshore currents, without being obstructed too abruptly by any configuration of natural or artificial nature, are able to build up a "bar bridge" to carry the sand transport.

The dimensionless parameter $\frac{M_{mean}}{Q_{max}}$ seems to be of significance for the by-passing procedure itself. The value of this ratio indicates whether by-passing is a predominantly "bar-bridge" or a predominantly "tidal flow" transfer, by which material is flushed out of the inlet by ebb currents carrying the material in downdrift direction.

Reference is made to Table 1. When Q_{max} is expressed in cu. yd./sec. and M_{mean} in cu. yd./year, a value of $\frac{M_{mean}}{Q_{max}} = r$ between 5 and 900 has been found for the inlets considered.

From the experience about by-passing at those inlets, the following rule may be used as a guide:

$r < 10 - 20$ indicates predominant tidal flow by-passing
$r > 200 - 300$ indicates predominant bar by-passing

That M_{mean} is small compared to Q_{max} does not necessarily mean that conditions are very advantageous or even ideal for tidal flow by-passing. A large Q_{max}—therefore a smaller $\frac{M_{mean}}{Q_{max}}$—may still mean unsatisfactory by-passing if the tidal flow is not utilized properly for by-passing.

The parameter $r = \frac{M_{mean}}{Q_{max}}$ also plays a role in the stability of tidal inlets. Reference is made to ref. (6), where it is mentioned that in alluvial material the size of the gorge of the inlet under given boundary conditions depends upon the size of the so-called stability shear stress along the wetted perimeter of the channel for the given conditions referring to springtide flow. The value of t_s depends on various variables such as friction factors, bottom material, shape of cross section, wave action, suspended load and littoral drift characteristics, fresh water outflow and time history.(6)

In case of navigation inlet the value of $\frac{M_{mean}}{Q_{max}}$ should be kept low to furnish sufficient depth in the channels across the outer and inner shoals. When the bottom of the inlet consists of rock the size of the gorge cannot adjust itself to its natural dimension in alluvial material. Velocities in such inlets may be much higher than at inlets in alluvial material and by-passing may for this reason be very inadequate because of strong currents which push the material out in the sea beyond the littoral drift zone so that it is lost as nourishment material for beaches in the surrounding area. An example of this is the Bakers Haulover Inlet on the southeast coast of Florida (Miami). Maximum current velocities reach 7 - 8 ft./sec. Model experiments by the Coastal Engineering Laboratory of the University of Florida have indicated how important improvements for navigation as well as beach erosion can be secured in an economical way.

The ratio $\frac{M_{max} - M_{mean}}{M_{mean}}$ describes the littoral drift irregularity which together with the $\frac{E_{wa}}{M_{mean}}$ ratio and the absolute values of M_{max} and M_{mean} describes the littoral drift conditions.

Table 1

Flow and Littoral Drift Characteristics

INLET	Q_{max} yd^3/sec. (Springtide Conditions)	M_{mean} yd^3/year (Order of Magnitude)	$r = \frac{M_{mean}}{Q_{max}}$
Amelandse Gat, Holland	36,600	10^6	28
Aveiro, Portugal	3,000	10^6	334
Big Pass, Florida	720	10^5	139
Brielse Mass, Holland (before closing)	2,740	10^6	365
Brouwershavense Gat, Holland	30,000	10^6	33
Calcasieu Pass, La.	2,600	10^5	38
East Pass, Florida	1,720	10^5	58
Eyerlandse Gat, Holland	19,000	10^6	53
Figueira Da Foz, Portugal	1,100	10^6	910
Fort Pierce Inlet, Fla.	3,700	1/4 . 10^6	68
Gasparilla Pass, Florida	910	10^5	110
Grays Harbor, Oregon	48,000	10^6	21
Haringvliet, Holland	25,000	10^6	40
Inlet of Texel, Holland	97,000	10^6	10
Inlet of Vlie, Holland	94,000	10^6	11
Longboat Pass, Florida	1,430	10^5	70
Mission Bay, California (before dredging)	1,130	10^5	88
Oosterschelde, Holland	100,000	10^6	10
Oregon Inlet, N. C.	3,500	3/4 . 10^6	215
Ponce de Leon Inlet, Fla.	1,450	1/2 . 10^6	345
Port Aransas, Texas	1,870	10^5	54
St. Augustine Inlet, Fla.	2,700	1/2 . 10^6	175
San Francisco, California	210,000	10^6	5
Scheveningen, Holland	sluices	3/4 . 10^6	- -
Thorsminde, Denmark	sluices	1/2 . 10^6	- -
Thyboron, Denmark	7,450	10^6	134
White Sands, Denmark	sluices	1/2 . 10^6	- -
Westerschelde, Holland	115,000	10^6	9

The smaller $\frac{M_{max} - M_{mean}}{M_{mean}}$ is the better conditions exist for satisfactory continuous by-passing under equal flow conditions. When this coefficient is large, the by-passing might still be adequate but irregular, with intermittent supply of sand to the downdrift beach.

Speaking of navigation, improved inlets may be designed to assist nature in by-passing by increasing the turbulences or by streamlined and less obstructive jetty improvements. If possible irregularities in the littoral drift magnitude should be equalized. At Thorsminde, Denmark (Fig. 8), groins have been built on the updrift side to regulate littoral drift; the updrift jetty has been extended to establish transfer in deeper water and drainage sluices have been built which are also useful for flushing the entrance if necessary. Navigation conditions are now rather satisfactory for boats of up to 6 ft. draft and natural by-passing of about 500,000 cu. yd./year takes place.

At White Sands, Denmark, model experiments carried out 35 years ago indicated the shape of entrance most useful for flow and for material by-passing, and sluices to regulate discharge and useful for flushing were built. Navigation for boats with up to 8 ft. draft as well as by-passing of about 500,000 cu. yd./year must be considered as generally satisfactory.

At Aveiro, Portugal, a long-term trial and error process resulted in a funnel-shaped entrance in which the curvature of the updrift jetty is particularly advantageous for by-passing. Model experiments with the inlet at Figueira Da Foz are in progress and a similar solution regarding inlet stability and by-passing is being sought. The present natural by-passing is reasonably satisfactory but greater depth is desired on the bar.

At a new harbor in Denmark at Hanstholm special measures for better by-passing are being sought in comprehensive model experiments now in progress at the Technical University.

No general "formula" or "theory" exists for proper design of inlets for by-passing, but if favorable natural conditions for by-passing exist this possibility can be tested by model experiments prior to construction. It will usually be difficult to avoid leeside erosion but improvements can be gained by using a streamlined jetty or guiding works on the downdrift side. This has proved favorable at several harbors.

If nature is not willing herself to establish by-passing, man may be able to help as demonstrated by model experiments with the harbor at Mucuripe in Brazil where a by-passing arrangement using long pipeline connections from one side of the harbor to the other is to be replaced by a smaller installation for pumping sand across the entrance of the port. A similar arrangement is under discussion for Santa Barbara in California in connection with construction of a downdrift breakwater or jetty. See ref. (18).

In regard to inlets at which tidal currents play an important role in by-passing, the abovementioned reveals that with the assistance of a considerable tidal flow there is a much better chance of developing a satisfactory by-passing arrangement if the flow is used in an intelligent way. The requirement is that deposits by flood currents in the inlet or entrance be removed by ebb currents and carried downdrift by wave action. With proper alignment of inlet jetties the situation can be favorably influenced so that adequate by-passing and sufficient depth for navigation go together.

Often by-passing and navigation requirements are contradictory, in particular when bar by-passing dominates. Strong ebb currents at otherwise good navigation inlets with considerable tidal flow may be very harmful by jetting

material far out into the ocean where it is lost for the shore. If it is difficult to decrease inlet current velocities, such inlets may be arranged so that material is deposited on bay shoals in such a way that ebb currents only to a minor extent are able to carry the material back to the sea. These bay shoals may then serve as a borrow areas for artificial nourishment of the downdrift side by hydraulic dredging. Examples of this are Bakers Haulover and Hillsboro Inlets in Florida where material is pumped from bay or inlet shoals over onto the downdrift side.

4. CONCLUSIONS

1. Analysis of the sand transfer at various inlets shows that in many cases adequate natural by-passing exists by which sand is transferred from the updrift to the downdrift side of the inlet.

2. Nature itself demonstrates two different methods of by-passing: bar by-passing, and tidal flow by-passing. Most cases of by-passing present combinations of these two general "principles".

Whether we have predominant bar by-passing or predominant by-passing by tidal flow action usually depends on the ratio between littoral drift and tidal flow.

If the predominant littoral drift (M_{mean}) is expressed in cubic units per year and the maximum tidal flow under springtide conditions in the same cubic units per second (Q_{max}), by-passing may be described by the ratio

$$\frac{M_{mean}}{Q_{max}} = r$$

$r > 200 - 300$ indicates predominant bar by-passing
$r < 10 - 20$ indicates predominant tidal flow by-passing

The more regularly the transport of material by moderate to heavy wave action takes place, the better. Long periods with little wave action, and therefore little suspended load transport, may develop spits in or even barriers across the inlet by beach drift and bed-load transport deposits. Continuous moderate to heavy turbulence seems always to be an advantage for adequate by-passing.

3. Natural by-passing occurs at improved as well as unimproved inlets. In the case of unimproved inlets, examples of adequate by-passing can be found with bar by-passing as well as tidal flow by-passing, and with combinations of these.

Inlets with predominant bar by-passing are usually unfavorable to navigation. In most cases only small crafts are able to use them and improvement by dredging is often not economically justifiable.

When navigation requirements are associated with the desire for natural by-passing, inlets with tidal flow transfer have the advantage over inlets with bar by-passing provided the flow is utilized properly.

With jetty-improved inlets, natural by-passing occurs in many instances on bars as well as by tidal flow action. In both cases heavy wave or swell action seems important for transportation of sand back to the shore downdrift of the inlet.

Conditions for bar by-passing are more favorable in this respect when the littoral drift is not too predominant. The contrary is true in case of tidal

flow by-passing where a neutral direction of drift may result in continuously expanding off shore shoals.

Groins on the updrift side built for equalization of the littoral drift toward the inlet and sluices in the inlet useful for flushing of the channel have proved advantageous for proper bar by-passing at some places.

In the case of tidal flow by-passing, the cleaning effect of the (often concentrated) ebb current and the desire for by-passing may be contradictory, yet optimum conditions for sand by-passing and navigation requirements may be obtained by proper alignment of the jetties securing spoiling of littoral drift material in the downdrift direction where waves may pick it up and bring it back to shore.

Where satisfactory by-passing conditions have been obtained they have either been the result of a long-term (usually expensive) trial and error process or have resulted from hydraulic model experiments.

In other cases a compromise with nature has been obtained in such a way that material is allowed to be deposited in sand traps on the updrift side or inside the updrift jetty or on bay shoals from which the material is pumped over on the downdrift side.

4. In many cases of jetty-improved inlets or entrances to accommodate vessels of greater draft, by-passing arrangements using mechanical means such as dredging from stationary or non-stationary pumping plants are a necessity. Man is then faced with the following two problems, both related to inlet configuration:

(a) the desirability of having an area well fitted for accumulation of sand coming from the updrift side and well fitted for pumping or dredging of the material accumulated

(b) the desirability of having an inlet configuration which at periods of maximum load of littoral drift material (when some material will escape the dredging or pumping plant), utilizes the inlet flow most adequately for flushing purposes—thereby possibly by-passing the sand deposited by the flood flow.

Regardless of the establishment of mechanical by-passing arrangements it should be remembered that an inlet first of all is a "hydraulic-mechanism" and the best inlets in existence are still those which were well conceived and well tested before they were actually born. Too many inlets failed because they were neither well thought nor tested before delivery.

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INVESTIGATION OF BLUFF RECESSION ALONG LAKE ERIE[a]

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ABSTRACT

A systematic approach to the analysis of bluff recession problems is presented. Three inter-related studies are normally required: beach development, toe protection, and stable slope development. An engineered solution to each of the three can lead to efficient stabilization of the bluff. An analysis of a typical problem east of Cleveland is included as an example.

INTRODUCTION

The gradual recession of the shore areas along the Great Lakes has presented a definite challenge to all those who have been confronted with the protection of the shore facilities. In spite of the technological experience amassed through the years, the problem still exists and continues to represent a considerable waste of an important natural resource. Rather than fighting the problem, corrections have commonly consisted of the relocation of threatened facilities farther inland as the more economical expedient, allowing the recession to continue unabated for another cycle. However, with the increasing demand for lake front property for industrial as well as recreational development, the employment of such an expedient is proving to be less and less desirable.

The problem of shore recession has been defined in several ways, depending upon the topography and the scope of the problem in a given area. Considerable emphasis has been placed upon the differentiation between two major components, beach erosion and bluff failure. Of the two, beach (or

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shore) erosion is perhaps the more widely used, and at times is used as the designation for the entire problem. Herein, beach erosion refers to the net removal of material from the zone effectively acted upon by wave action. In accordance with the terminology of the Beach Erosion Board,(1) the zone extends from the low-water mark to the high-water mark or to the base of the cliff or bluff where present.

Bluff failures, on the other hand, refer to the displacement of material from the bluff proper and are similar in their characteristics to other types of natural slope movements. The only difference lies in the fact that additional forces induced by wave and ice action are present.

Most of the literature in the field of coastal engineering has been more concerned with investigations of beach erosion phenomena than with bluff stability. Perhaps this is because beach erosion is based upon hydraulics while bluff stability is of more concern to the field of soil mechanics and geology. To some extent, the philosophy has appeared to prevail that a stable beach will also serve as the stabilizing factor which is necessary to maintain stability of the bluff materials above. One implication of such a philosophy is that the wave energy becomes expended upon the beach rather than upon the toe of the bluff. In any case, the majority of protective works has been concerned with such structures as are necessary for adequate beaches or, where beach building is difficult, structures to withstand the wave attack.

Perhaps in some instances so much emphasis has been placed upon beach stability that the elements of bluff security have been neglected. In many areas where the toe of the bluff is protected from wave action by a beach, bluff failures still result and still present a serious problem. Thus, other forces do contribute to instability and must be considered. It is an ironic fact that where nourishment of beaches is natural (not artificial), receding of the bluff areas and the beaches must occur elsewhere in order to provide the beach-forming material. Consequently, relying upon the beach as a protection for the toe may still lead to undercutting and bluff failure, even if proper measures are taken to insure stability of the bluff. As land along a lake becomes more and more at premium, natural beach nourishment will undoubtedly become less and less tolerable. More desirable will be the methods which provide a more permanent protection, not so susceptible to the whims of nature.

The basis for the paper presented herein stems from a study recently conducted by the authors, and sponsored by the Ohio Division of Shore Erosion. The study attempted to focus the role that bluff stability plays in the over-all recession problem.(2) Problems related to beach stability were also considered but only in those instances where they directly affected bluff stability.

The primary purpose of this paper is to delineate and discuss principles and a method of approach which can be applied to an analysis of bluff recession problems. Included will be those established principles associated with conventional slope stability analyses. The utilization of several of these principles is demonstrated in the analysis of the bluff recession at Perry Township Park in Northern Ohio.

The terminology used is, in general, in accordance with that recommended by the Beach Erosion Board (Fig. 1). It was felt two exceptions are the terms shore zone and lake front which were considered as more applicable (and better understood) for lake conditions than would be coastal area and coast, respectively.

Fundamental Characteristics of Natural Bluff Failures

Since the mechanics of bluff failures apparently are no different from those associated with other slope failures, the two are herein used interchangeably. Even though the additional variables, wave and ice action, can be quite significant, their presence can be adequately treated by the mechanics of conventional slope stability analyses. Consequently, reasons for differentiating between the two are not related to the mechanics involved and thus such segregation is not too meaningful. Pursuing the problem in this manner appears logical, and offers the distinct advantage in that the already established classification and analytical methods pertaining to landslides can be directly employed.

Description of Bluff Failures

In general, the downslope movement of earth materials can be divided into two broad categories insofar as mechanics are concerned: (1) landslides, and (2) erosion. Landslides are denoted as downward and outward movements of slope-forming materials by falling, sliding, or flowing types of action.[3] Specific nomenclature for landslides with varying materials, moisture content, and speed of movement as summarized elsewhere are shown in Table 1.[3] Frequently, materials in transit are intact masses at the start of movement, but disintegrate into considerably smaller masses as movement progresses, and may develop into a flow of material which resembles the movement of a viscous liquid.

Erosion can be defined as net removal of particulate material which occurs as a result of the hydraulic action (fluviraption) of running water.[4] It is characterized by a surficial activity in which individual grains are loosened and removed by the stream of water (or wind). Thus, while the differentiation between erosion and landslides may, on occasion, be one of degree, landslides will include all cases of earth transport except those in which the major driving force is derived from the energy of a moving stream of water (or air) or water - soil mixture.

Erosion is not conventionally analyzed as a slope stability or earth-shear problem. Rather, the resistance of the earth material to rapidly moving water is considered empirically and if an erosive material is involved the

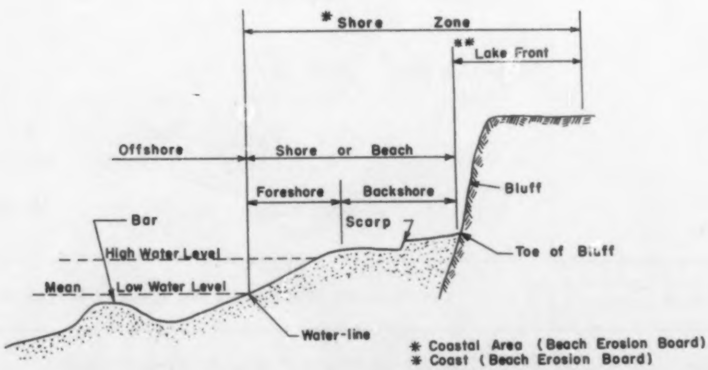


Figure 1. Beach and bluff terminology

designs are based upon (1) flattening the slope, (2) channelizing flow along a protected path, or (3) diverting the surface runoff.

Landslides have been classified in many ways.[5,6,7] The most recent system based upon the mechanics of failure,[2] recognizes three basic types; i.e., falls, slides, and flows (Fig. 2). They are defined as follows: "In rockfall and soilfall, the moving mass travels mostly through the air by free fall, leaping, bounding, or rolling, with little or no interaction between one and another. In true slides, the movement results from shear failure along one or several surfaces, which are visible or may reasonably be inferred. In flows, the movement within the displaced mass is such that the form taken by the moving material or the apparent distribution of velocities and displacements resembles those of viscous liquids".[2] From a stability analysis viewpoint, slides and some types of flows have been frequently considered. Falls are generally the result of tension failures and stress analyses have rarely been conducted. For plastic flows, considerably more knowledge of the action of earth material stress to the plastic range is required. For viscous-type

TABLE I

*CLASSIFICATION OF LANDSLIDES

Type of Movement	Type of Material			
	Bedrock		Soils	
1. FALLS	Rockfall		Soilfall	
2. SLIDES				
Few Units	Rotational Slump	Planar Block Glide	Planar Block Glide	Rotational Block Slump
Many Units		Rockslide	Debris Slide	Failure by Lateral Spreading
3. FLOWS	All Unconsolidated			
	Rock Fragments	Sand or silt	Mixed	Mostly Plastic
Dry	Rock Fragment Flow	Sand Run; Loess Flow		
		Rapid Earthflow	Debris Avalanche	Slow Earthflow
Wet		Sand or Silt Flow	Debris Flow	Mudflow
4. Complex	Combinations of Materials and Types of Movement			

* Taken from "Landslides and Engineering Practice" Highway Research Board, Special Report 29.

flows the inherently high water contents have the adverse effect of decreasing the shearing resistance of the materials to a point at which the movement resembles and performs as a liquid, a condition associated with negligible shearing resistance.

Environmental Influences on Slope Failures

The recognition of the environmental variables and their rational expression in terms consistent with those employed in stability analyses is of fundamental significance in the study of earth movement. Slope failures can be considered as the visible or outward manifestations of the constituents of a given slope, undergoing adjustment to their environment. In every case of failure, the adjustment can be attributed to either an increase in stress or a decrease in shearing resistance or both. It follows that since the environmental variables (physical and chemical processes) responsible for disrupting the stress equilibrium are subject to considerable variations, the rates of adjustment will vary accordingly.

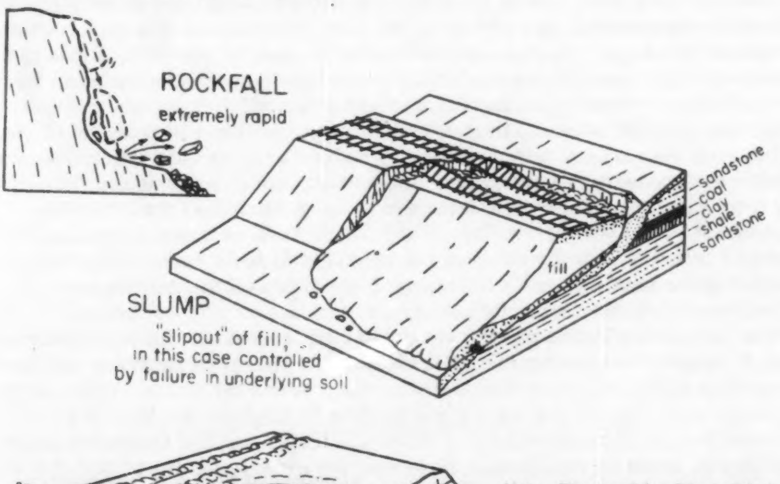


Figure 2.

After "Landslides and Engineering Practice" - HRB Committee on Landslide Investigations

According to the concept of uniformitarianism, a fundamental test of geology expressed by Hutton in 1785, there is no reason for the phenomena associated with slope failures to be materially different today than they have been at any time during the existence of the slope. The concept states that the same natural physical and chemical processes and laws operating today have operated throughout geologic time, although not necessarily with the same intensities. This implies that bluff recession has continued throughout geologic time, and is one of varying degree, the magnitude of which depends upon the size of the unbalanced forces. Considering the earth as a whole, the concept has been generally accepted. For specific, localized areas, all forces present during a specific time interval, are not now present and the concept is not totally relevant; i.e., the glacial age and the Lake Erie shoreline. Within the shorter time period comparable to the life of an engineered structure, the concept is more valid. The recession of the shore areas along Lake Erie was mentioned in the diaries of some of the earliest missionaries in the area. An instrument survey report(4) made in 1906 mentioned the fact that the reason for the survey was to establish the parts of a highway destroyed and injured by the washing and sliding of the land within said highway, occasioned by natural drainage. Because no mention was made of any violent cataclysm occurring, inference can be made that the soil movement is similar to that present today, where a gradual but unceasing loss of soil occurs.

No attempt will be made here to relate the activities with respect to slope stability of the various environmental variables such as rain impact, runoff, weathering processes, frost action, subsurface water, wave action, lake level, and free ice. The reader is referred to other publications for such discussions.(2,9,10,11,12,13,14,15,16,17,18,19) It is to be noted, however, that a lack of a better understanding of the behavior of soils under the influence of many of these environmental influences has prevented the development of more completely rational relationships.

The influence of wave and ice action on bluff recession can be considered from a viewpoint of mechanics. Essentially the energy of the wave and ice is directed initially in a shoreward or near-shoreward direction. Thus, during one stage of the cycle, the forces are tending to improve the stability. If it were not for the characteristics of wave and ice during the recession phase, their effect would be beneficial. However, the erosive nature of both the receding stage and the lateral component of littoral currents provide the major source of concern. It is true that structures at the toe can be severely stressed by the shoreward wave and ice action, particularly if not placed directly against the bluff. Therefore, such structures must consider the level of stresses imposed, whereas natural bluff materials will tend to be (1) compacted or (2) well able to resist the compressive forces applied.

Effect of Type of Bluff Material

The material present in the bluff will control, to a great extent, the rate of recession as well as the corrective techniques utilized in the elimination of the problem. Proceeding under the assumption that either a landslide or an erosion problem is involved when bluff recession is encountered, the principles applied to the two mentioned phenomena are of interest. Certainly the shearing resistance and the resistance to weathering are key factors.

Engineers conventionally divide natural materials into bedrock and soil. To the geologist, this delineation is a source of considerable concern, for he

sees all natural materials not in motion as being consolidated, and the question is purely one of degree. Thus, a recent alluvian would normally be classed as poorly consolidated in comparison to a glacial till. In turn, a glacial till may be well-consolidated in comparison to other recent deposits, but not with reference to some Pennsylvanian sandstrains. Bedrock, to most engineers, means an ancient rock which has not been transported nor completely weathered in place since its first development into a rock. Even though glacial tills and other well-consolidated soil masses may be well on their way to being "rocks," engineers still normally refer to them as soils.

When encountered on exposed slopes, the significant difference between bedrock and soils is normally their shearing resistance. Thus, the engineer's conventional interpretation rarely gets him into trouble, since bedrock shear values are generally much higher than required for stability against shear even with near-vertical exposures. Such is not the case for soils, and the resistance to shear can be so poor as to require slopes as flat as 15:1 (horizontal:vertical).

There are occasions when the engineer's interpretation is misleading, and these are associated most frequently with weak shales (with low shearing resistance) and well-consolidated soil layers (with high shearing resistance). It is certain that the terms well-consolidated and poorly-consolidated are more appropriate from a classical viewpoint. However, engineering practice is such that the terms "bedrock" and "soil" are more desirable and emphasis, therefore, must be placed on the border line conditions with relation to shear properties.

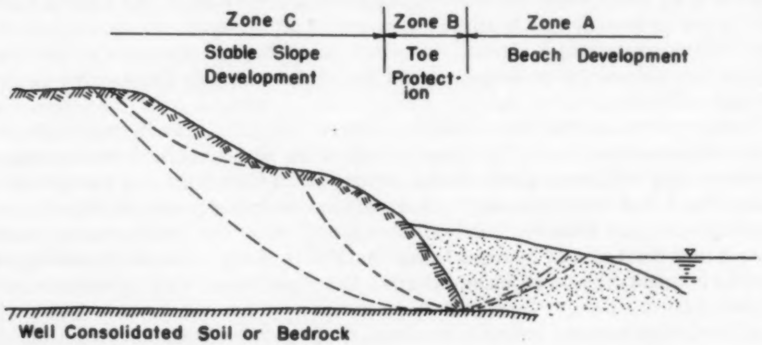
In bluffs the normal bedrock materials are generally safe against shear through the strata and must be analyzed from a viewpoint of resistance to weathering and erosion. Soils, on the other hand, must be considered as being susceptible to shear through the layer, as well as subject to weathering and erosive conditions. The additional complexities of bedrock shear along sloping strata and the flow of soil masses (such as spontaneous liquefaction) fit these preceding generalities, and constitute refinements rather than fundamentally different behavior.

General Analysis of Bluff Recession Problems

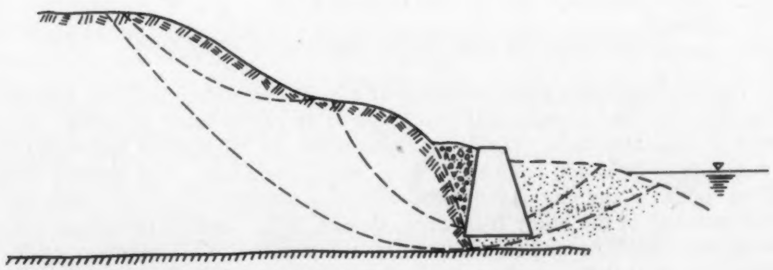
A rational approach to the solution of a bluff recession problem may be delineated into the conventional five steps of an engineering analysis: (1) collection of available data, (2) field investigation, (3) laboratory investigation, (4) analysis of field and laboratory data, and (5) solution. The degree of effort expended on any one of the steps will be a function of factors such as: (1) the purpose of the study (research, design, etc.), and (2) economic considerations. Bluff recession studies are comparable to all engineering investigations in that the conduct of all phases is related to the type of problem involved. Thus, studies of bedrock conditions will vary from those concerned with unconsolidated sediments. Another example is the difference between the economic considerations associated with the investigation of a million-dollar structure and an inexpensive one. It is apparent that all solutions will not require the same detailed investigational and analytical effort. Of the five major steps designated above only the last will receive specific consideration in this paper. Valuable suggestions as to field studies are contained in the recent test on landslides[2] and in the classic work on subsurface exploration.[20]

Components of the Problem

A bluff recession problem will consist of three inter-related components: (a) beach provision, (b) toe protection, and (c) stable slope development. For purposes of this study the components are referred to the zones of activity illustrated in Fig. 3, Zones A, B, and C, respectively. Beach provision means simply that a sand beach is required for recreational or other purposes. Toe protection is the resistance required at the toe of the bluff to protect the slope from the undermining and erosive action of the lake. The development of a stable slope is concerned with the area above the level affected by lake activity. The solution to a single bluff erosion problem may incorporate a



Case A – Protective beach present or to be provided – toe protection provided by beach – conventional slope stability problem.



Case B – Non-protective or no beach present or to be provided – toe protection structure required – conventional slope stability problem.

Figure 3. The three components of the bluff stability problem

measure which will meet two, or even all three, of the types of requirements. However, for economic reasons and in order to analyze the effect of a solution, the three basic components should be considered in the above stated order.

In many areas, particularly public and residential, recreational beach facilities are desired and the development of a sand beach may be incorporated into the contemplated measures for providing bluff stability. As a result, toe protection of the bluff from erosional undercutting by wave action is provided if the beach is sufficiently extensive. The added weight of the beach material may or may not significantly increase the shearing resistance and improve the stability of the bluff against deep-seated shear failures. If beaches are relied upon to provide the necessary toe protection, it becomes critically important to provide the structures necessary to insure the continuity of the required beaches, particularly during storm periods. In other words, the stability of the bluff as it is affected by toe erosion should not be jeopardized by relying upon a beach development which is subject to the reversals produced by nature.

1. Beach Development.—Shown in Fig. 3 and in Table 2 are the requirements of each component of the shore zone which result from a given type of beach development. The beaches which do provide permanent year-round protection for the toe are defined herein as protective beaches.

Non-protective beaches refer to those for which only a limited amount of toe protection may be available from beach-forming material. Such conditions develop because of (1) frequent cycles of erosions and accretion, (2) insufficient beach forming material, or (3) high cost of providing constant artificial

TABLE 2

VARIOUS REQUIREMENTS OF THE THREE BLUFF STABILITY COMPONENTS

Zone A Beach Development	Zone B Toe Protection	Zone C Stable Slope Development
Case A. Protective Beach 1. Sufficient beach to dissipate the energy, prior to its reaching the toe of the bluff.	1. None - provided by beach.	1a. Stability against toe and slope shear failures (some resistance to toe failure provided by beach) 1b. Protection against erosion and weathering processes (surface runoff, rain impact, seepage, and frost action).
Case B. Non Protective Beach 2. Beach required but permanency less important than for a protective beach.	2. Partially by beach and toe structure	2a. Stability against toe and slope shear failures (some resistance to toe failure provided by beach and toe structure). 2b. Protection against erosion and weathering processes (surface runoff, rain impact, seepage, and frost action).
3. No beach required	3. Toe Structure required	3a. Same as 2a. above except that no resistance to toe failure can be assumed to be provided by beach. 3b. Same as 2b above.

nourishment. For other than protective beaches, a structure will be required to provide either supplemental or total toe protection. For example, a combination of an offshore sea-wall (or an enclosed breakwater structure) and artificial nourishment will provide reliable toe protection as well as dependable beach facilities for recreational purposes. If the provision of the protective beach is too costly and the recreational aspects are not too important, structures at the bluff may be required to provide protection from wave and ice attack or to restrain the movement of the bluff materials. Many of the restraining structures described in detail in the landslide treatise(2) may also be employed with proper modifications to provide protection from wave and ice action. Most notable of these include rock buttresses, concrete retaining walls, sheet piling, and combinations thereof.

2. Toe Protection.—Toe protection should be designed for the specific purpose of protecting the slope against erosion or other detrimental influence of the lake. The two principal factors to be considered are wave action and ice activity. In a general way, it can be stated that the toe area attacked by the lake must be protected by materials which will not (1) erode under wave energy, (2) be fractured nor displaced by wave forces, (3) disintegrate rapidly under alternating freeze-thaw conditions, and (4) fail under impact and direct forces produced by ice accumulation.

A toe structure at lake level is frequently utilized for accomplishing slope stability. In such cases, toe protection from the lake is combined with the development of a stable slope.

3. Stable Slope Development.—In a rational approach to slope stability analyses of unconsolidated materials, the stress-strain relationships necessary to provide equilibrium must be understood. Natural or man-made factors, both detrimental and beneficial, must be evaluated in terms of the changes in the stress-strain relationships. Involved in the analysis of soil slope failures are the following steps:

1. Recognition of the environmental influences present at the time of failure and an estimate of their magnitudes in terms of stress components.
2. Estimate of the shearing strength of the materials and subsequent expression in computing the resistance to stress components.
3. Resolution of all force components present at time of incipient failure or equilibrium just prior to failure.
4. Adjustment of values obtained in Steps 1 and 2 if unbalanced forces remain after Step 3.

Detailed procedures for conducting slope stability analyses are contained in several tests[3,7,21,22,23] and are not repeated in this report. The difficulties and uncertainties in obtaining data imposed limitations on the reliability of the results from quantitative analyses. However, in the absence of analyses on a quantitative basis, only experience and empirical rules are available. The degree of accuracy of the empiricism is quite variable and almost always unpredictable.

Slope stability for a given area can be enhanced by an analysis of the slope problems which have developed in the area under consideration. Analyzed on the basis of the mechanics of the movements involved, slopes which have failed can provide more reliable data with respect to stress-strain relationships than can laboratory-conducted strength tests. However, care must be exercised in the application of shear data developed from slope failures.

Normally, shear data derived from a failed slope should only be applied to an analysis of that same slope.

Many measures may be taken to improve the overall stability and decrease the design forces. Among the more widely used techniques discussed in great detail in the literature(2,7,23) are the following:

1. Slope excavation through
   a) removal of material at the top of the bluff
   b) removal of unstable material and replacement with more suitable material
   c) flattening of slopes
   d) benching of slopes
2. Surface and subsurface drainage by
   a) surface diversion ditches
   b) surface slope treatment
   c) sealing of cracks and fissures
   d) regrading of potential water collecting areas
   e) horizontal drains, tunnels, and trenches
3. Retaining Structures
   a) buttresses
   b) cribs or retaining walls
   c) piling

The effects of these techniques can be estimated and the benefit derived can be evaluated in terms of their beneficial influence on either the shearing resistance or the shearing stresses. Their proper inclusion in the stability analyses will indicate the value and extent of the corrective measure.

When well-consolidated, bedrock material is exposed in the bluff, the rate of recession is normally quite low, and the analyses required are different than for soils. The conventional slump or circular arc shear failure do not occur except for very high slopes or in very weak shales. Shear failures along bedding planes or other tilted discontinuities do require consideration. Methods for estimating stress and shear relations are discussed in landslide texts.(3)

Bedrock will more frequently be susceptible to weathering and erosion than to massive shear failures. Differential (varied rate) weathering can be of specific concern, particularly if the weaker layer underlays the more resistant, and an undermining condition develops. Also, presence of joints, cracks, and other discontinuities can produce considerable rock fall. For well-consolidated soils, and for relatively low bluffs, weathering and erosion may be more critical than shear. As can be seen, then, there is a borderline area between bedrock and soil materials for which the conventional engineering classification can be misleading.

For weathering considerations, berms or benches are most commonly used to minimize maintenance and damage. Lake bluff problems do not have as serious a hazard condition unless a well-populated beach or shore structure is at the toe. However, to minimize the recession, benches can be of value.

The design of rock slopes for engineering works has been described in detail.(3,13,14) Essentially, four basic types of design are currently in use: (1) uniform slope, (2) variable angle, (3) maintained bench, (4) non-maintained bench (Fig. 4). The first is most frequently used on small vertical slopes, or

when the material is quite uniform, with few discontinuities. The remaining three are theoretically sound techniques, if sufficient information is available. In the case of lake bluffs, the layers present are relatively easy to identify. The rate of weathering and presence or absence of joints and cracks require more careful study. With such information the type of design is controlled by economics and esthetics. For the variable angle and the maintained bench procedures, the various rock layers are excavated to a slope that is designed to produce little or no debris due to weathering. The bench design is essentially the same as the variable angle except that the benches provide for some margin of error or as an interceptor for large rock fragments that develop from joints, cracks, etc.

The non-maintained slope is the most economical but does not give the best appearance for several years. The various rock layers are cut to a slope more nearly vertical than that to which the strata would normally be expected to weather. The immediate weathering produces rock debris on the bench. If the debris is not removed, it protects part of the rock slope from

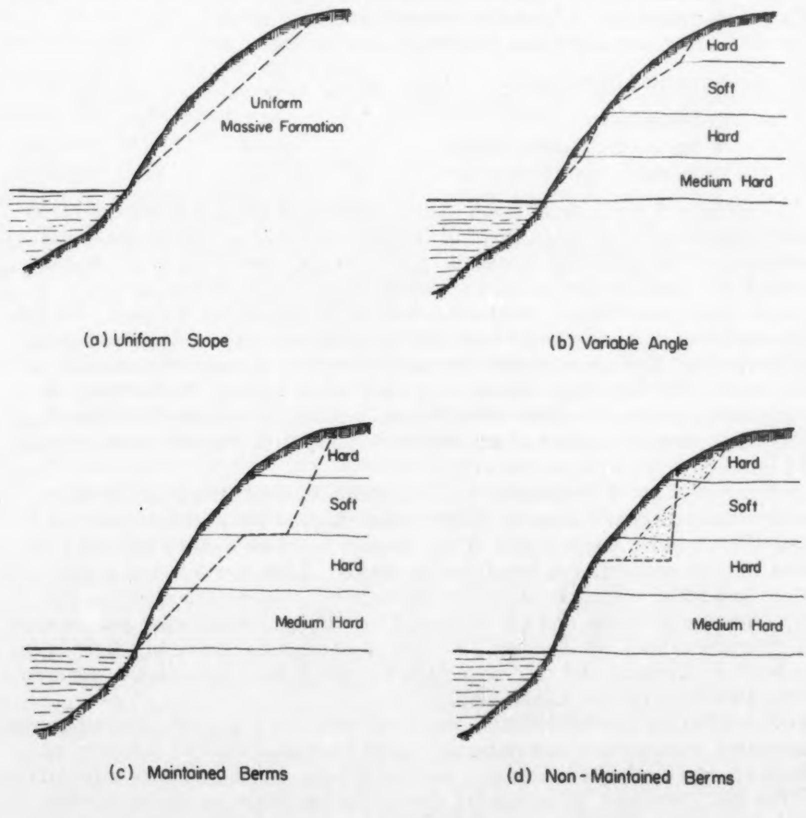


Fig. 4 — Four Basic Types of Rock Slope Design

further attacks by climatic variations. This technique normally results in an overall steep slope and less property affected.

The choice of slopes, width of benches, and height between benches are a function of the rate of weathering. Experience and observations with the given rock are the best guides. Some general suggestions as to values are given in the previous mentioned text on landslides.[3]

Detrimental slope factors, such as frost action, in material highly susceptible to erosion by rain impact, runoff, and seepage must be considered in the design. Although drainage structures have often minimized frost action and seepage, they are a maintenance-type of correction in that continuous observations must be made of the effectiveness of drains. Solutions of a more permanent nature involve the removal of poor soil and replacement with a more suitable material as a protective blanket. Seeding of slopes and proper surface drainage provide some protection from rain impact and runoff.

Economics

For a given bluff stability problem there will be a number of possible corrective measures which are adequate or equivalent for the technical requirements. The choice between the alternates will be dictated by economic considerations. In order to have an initial basis for comparison, the several possible solutions are designed with approximately the same factor of safety so as to become technical equivalents, i.e., equally capable of fulfilling the purpose expressed in the design requirements. At this point, the analysis is no longer concerned with the technical aspects but deals with the economic considerations.

The economics must be long range in their perspective and include maintenance and amortization costs as well as the capital investment. Reduction to an annual cost basis provides a means of determining the most economical solution, namely, the one with the lowest annual cost. Certain measures require high first costs and low maintenance costs. Other methods entail exactly the opposite characteristics, and yet the two may be technically equivalent. When small government units or individual property owners are involved, the availability of funds may be a limiting factor and a solution requiring a low capital cost may be more attractive or even required. In all situations, however, the various costs should be reduced to an annual cost in order to demonstrate whether the available funds can produce an effect commensurate with the expenditure.

The preceding discussion presumes that some action is necessary for the abatement of bluff recession. However, if some question arises as to the justification of any corrective measure, a benefit-cost (or-rate-of-return) type of analysis may be required. For a given treatment, such a study involves a comparison of the benefits derived and the costs required. The ratio sufficient to produce correction depends largely upon some minimum value, arbitrarily dictated by policy. Where the benefits derived may vary with the type of corrective measure contemplated, the extension of the annual cost basis will be necessary so as to include the effect of benefits derived. Such a situation presents itself in the comparison of a protective beach with a nonprotective beach. Although at times the beach associated with the latter type solution may be adequate to meet the needs of the area, the fact that it is not as dependable as the former is reason enough to expect smaller derived benefits. The placement of monetary values in such instances may prove quite

difficult, and the evaluation of benefits should be attempted only if, after the consideration of other factors, there is little difference in the alternate solutions.

Considerable economic benefits can be obtained if the planning of the protective works involves a number of adjacent bluff areas. The expenditures required to tie the structures into the bluff to prevent flanking can be decreased measurably. The financial status of a larger group is more sound, and it is generally granted that cooperative efforts are ideal and that an organization with some authority is needed to coordinate the activities.

Analysis of Corrective Actions

The preceding sections have presented a general discussion of the theories which are applicable to bluff recession analyses. For the specific problem at Perry Township Park, detailed information was obtained on the geology, climatological data, and waves and lake level variation. In addition, a topographic survey using photogrammetric procedures was used to measure loss of soil from the slope; vane shear tests as well as laboratory soil tests were conducted. Ground water and frost penetration data were obtained, the latter using specially designed simple equipment.

For purposes of illustrating the method of analysis, bluff recession at Perry Township Park (Fig. 5) will be used in the following pages. While the type of problem encountered is no doubt typical of others along Lake Erie, specific details of the study are not intended to be applicable elsewhere. The principles, methods, and techniques employed, however, will find application in other areas.

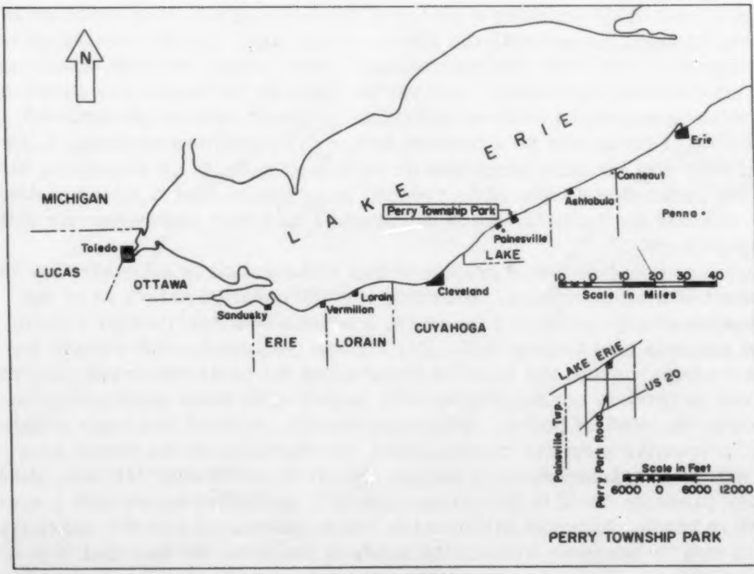


Figure 5. Locality Map

Perry Township Park is a publicly-owned recreational area, and terminates lakeward with bluffs averaging approximately 40 feet in height (Fig. 6). Geologically speaking, the bluff deposits consist primarily of glacial till, underlain at a depth of eight feet below normal lake level by bedrock shale and capped by lacustrine deposits (Fig. 7). Soil movement was observed to be predominantly erosional (Fig. 8). Slump-type shear failures were not observed. Ground surface profiles obtained from topographic data of the period between 1946 and 1958 indicate an average annual soil loss (measured horizontally) of approximately three feet. It was concluded that four natural processes are contributing significantly to the bluff recession: (a) frost action, (b) groundwater and seepage forces, (c) sheet and concentrated runoff, and (d) wave action.

As will be true in most bluff problems, it became obvious that overall protection required (1) adequate protection of the toe from wave and ice action, and (2) stable slope development with respect to landslide and erosion processes. Four different courses appeared possible: (1) stable slope development with toe protection, (2) toe protection without provisions for stable slope development, (3) stable slope development without provisions for toe protection, or (4) no corrective action. Of the four, only the first mentioned contains both of the requirements for a permanent correction. The second procedure constitutes an alternate which tends to reduce the problem for the moment, while the last two would not, of course, affect the rate of recession. While any of the four courses are possible choices of the administrative group responsible for the Park, the last-mentioned is not of particular design interest. The listing of "stable slope development without toe protection" was made to emphasize that such an alternate might be selected. However, in the final analysis, either of the last two will have a highly comparable influence on bluff recession; i.e., the rate will not be affected. In effect, then, no expenditure to develop a stable slope without protecting the toe can be effective. Consideration of the first two alternates will serve to illustrate the quantitative nature of the analyses.

1. Toe Protection with Stable Slope Development.—Since the area in question is a public park, development of beach facilities for toe protection and other purposes appears desirable if within economic limits. Consequently, major consideration should be given to the presence of a beach and to its influence on the structural and economic controls for the maintenance of bluff stability. The analysis is divided into the three basic components; i.e., beach provision, toe protection, and stable slope development.

a) Beach Provision. A choice exists between the type of beach desired, protective versus non-protective. The final decision involves economics, including the extent to which recreational facilities are desired, and esthetics. With respect to economics and to bluff recession the cost of developing and maintaining a permanent beach should be compared to the cost involved with constructing a toe structures which would be necessary in conjunction with a non-protective beach. While initial costs will undoubtedly favor a non-protective beach, the esthetics and the absence of a permanent beach could be important factors. The preliminary analyses were conducted without regard to initial costs nor the ability of the agency to make a sizable capital investment. In any given situation, considerations of alternate solutions will often be restricted to those techniques which are economically feasible.

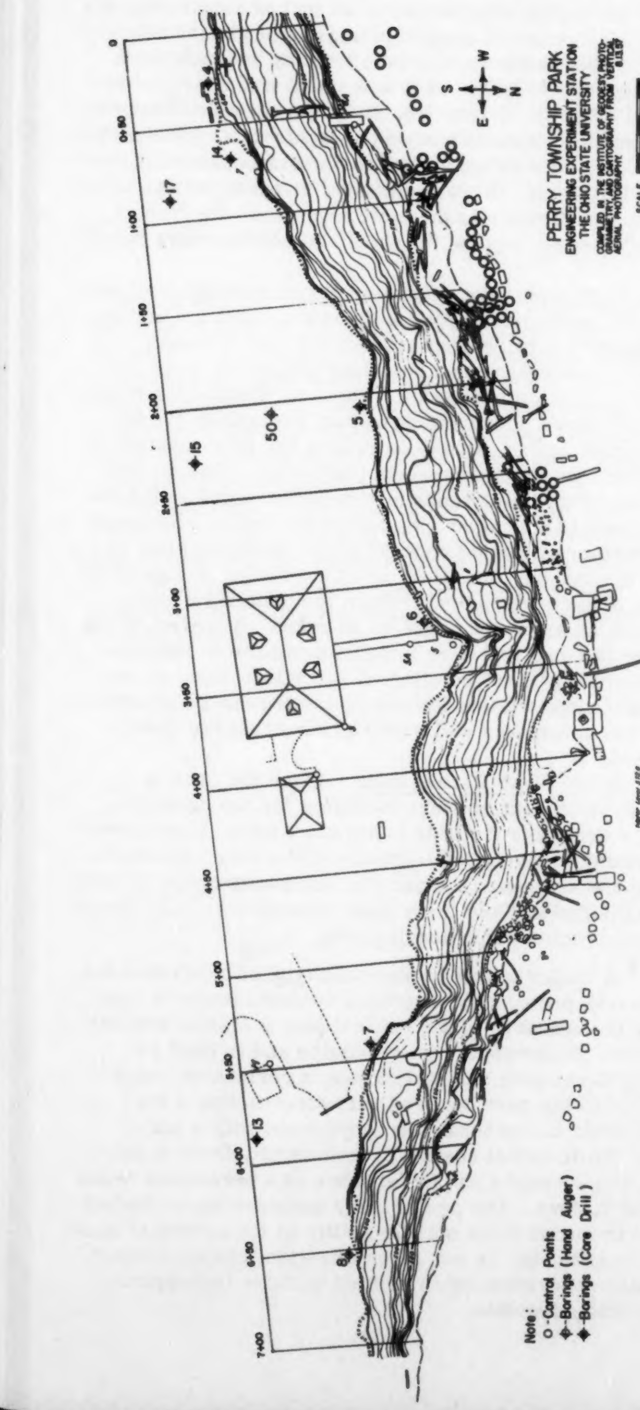


Figure 6. Topographic Map Compiled From Airphoto Coverage.

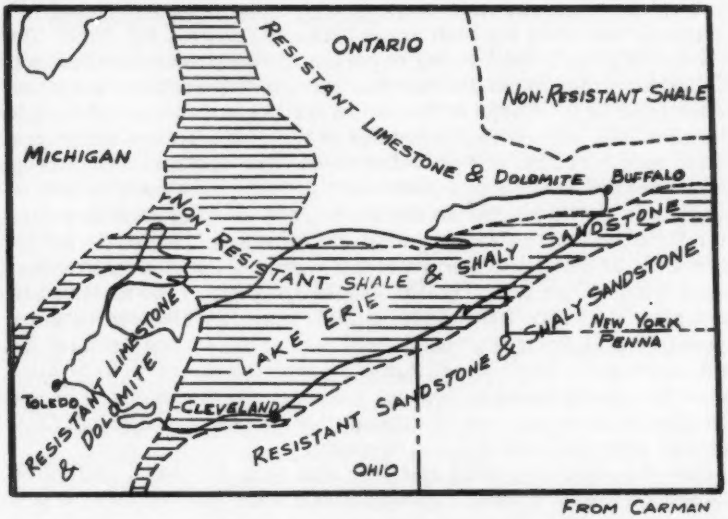


Figure 7a. The Distribution of Rock Types in the Lake Erie Region.

Figure 7b. Geologic Profile at Perry Township Park.

According to the cooperative beach erosion study,(24) the breakwaters at Fairport and Ashtabula, Ohio, have significantly decreased the amount of beachforming material transported by littoral currents from the Fairport and Ashtabula areas, west and east, respectively, to the area under study. It was further stated that since the bluff areas between Fairport and Perry Township Park contained little beachforming material, natural beach development is not feasible for areas located within that stretch of shoreline. A system of groins designed by the Corps of Engineers for the area depended completely upon the continued but controlled erosion of its bluff area and the trapping of the limited supply of sand and gravel present. The contention in the report was that controlled erosion was sufficient for beach development, and was within tolerable limits insofar as threatening the existing park structures.

It is difficult, on the basis of present-day knowledge, to design for complete cessation of soil movement from the bluffs; and designing to some fixed loss is not feasible, particularly if the zones from which the material is to come are also to be specified. Thus, natural beach development is not a reliable possibility at Perry Township Park, and artificial nourishment must be accepted as the most feasible solution if a protective beach is to be utilized. This could be accomplished by several methods: (1) placement of beach material in piles to be acted upon by natural processes for scattering, (2) direct placement of material, and (3) sand by-passing.

An off-shore structure would also be required to provide a protective beach at Perry Township Park. Such an installation should conform to the Beach Erosion Board's recommendations as to elevation; i.e., the structure should be at least eight feet above low water datum (an elevation of 578.5 feet at the shoreward end). With such a structure, the beach surface can be permanent without undue additional difficulty and expense for maintenance.

For a non-protective beach, there are special problems associated with developing a smaller, non-permanent sandy area. Solutions of this type include methods other than off-shore structures.

From the standpoint of eliminating bluff recession, the methods of obtaining non-protective beaches are immaterial. In fact, the decision to have or not to

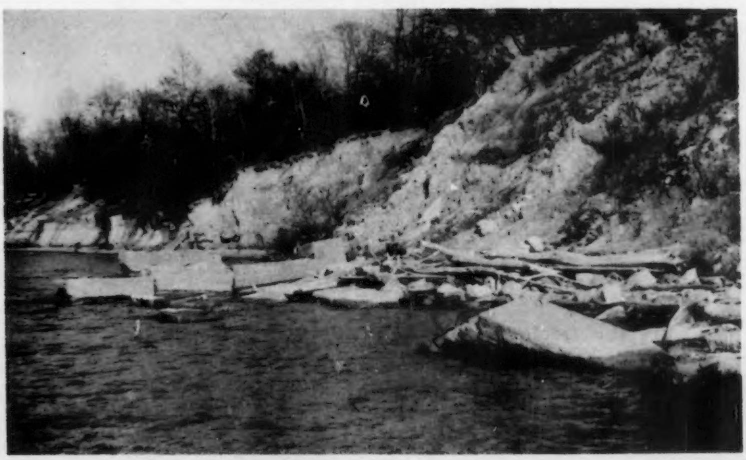


Figure 8. Bluff Erosion at Perry Township Park, Ohio.

have such beaches is more in the realm of an administrative decision based upon hydraulic considerations. For bluff recession the importance of the designation of "non-protective beaches" is the parallel requirement for some other form of permanent toe protection.

b) Toe Protection. The provision of toe protection has been described as a function of both the type of beach provided and the slope stability requirements. Furthermore, with a protective beach no toe protection against wave action is required since, by definition of such beaches, significant wave energy is dissipated prior to reaching the toe of the bluff. For non-protective beaches one must design for the condition of the toe of the bluff exposed to wave activity. Insofar as slope stability is concerned, there are occasions when in the interest of economy, the toe protection can be combined with the solution to the slope problem. Such considerations will be treated in the subsequent section on "Stable Slope Development."

One of the more difficult decisions involved in the design of a given toe protection structures is the determination of its proper height. Other factors meriting consideration include: (1) the design wave height and its frequency of occurrence and duration, (2) the extent and probability of overtopping and wave run-up, (3) damage encountered as a result of overtopping and wave run-up, (4) short-term fluctuations of lake level and their frequency of occurence, (5) probability of concurrent occurrence of high lake level and design wave, (6) depth of scour, (7) effect of ice action, and (8) economics.

The design of the toe structure must first meet the requirements imposed by the forces which develop from wave and ice action. Having satisfied such requirements, the stability of the toe structure must be analyzed for the stress conditions imposed by the bluffs. The principles utilized in designing devices for shore protection against wave and ice have been described.(19) For the Perry Township Park the considerations for toe protection are included in the following section.

c) Stable Slope Development. From a series of stability analyses conducted in order to account for all potentially significant slip surfaces, an adequate slope design can be obtained. Helpful in determining the location of the critical slip surface are such indicators as (1) relatively weak or strong soil layers and their locations within the slope, (2) combined height and angle of inclination factors which may induce mid-slope as well as toe or base failures, and (3) obvious boundary conditions provided by bedrock, benches, etc.

The problem of slope stability at Perry Township Park involves three vertical levels as described in Fig. 3; i.e., overall slope or base failure (zones A to C), toe slope (zones A and B), and the upper slope (zone A). The consolidated till stratum, which constitutes most of the material in the bluff, is firm and has a high shearing strength. Therefore, overall slope and toe-of-slope failures cannot develop for the height of bluff involved. Stability analyses using the Swedish Circle method indicate a safety factor of nearly three for the lowest shearing strength obtained by tests, and for the most severe pore pressure conditions to be anticipated.

Concluding, then, that shear failures will not extend through the till stratum, the slope stability problem is concerned with (1) the weathering or erosion of the till stratum which lies above the toe protection, and (2) the

bluff above the till stratum. With regard to the first of these two problems, only about 6.5 feet of the glacial till will lie above the uppermost level of toe protection. Weathering and erosion control can be achieved by sloping the till stratum to a 1/2:1 (horizontal: vertical) slope with a small bench at the top of the stratum to prevent the undermining of the upper bluff material. The dimensions of the bench depend upon the theory of design to be employed,(3) assuming that the bench will not be a depository for the upper bluff material, the width of the bench need only be one to two feet. A larger dimension may be more economical from a construction viewpoint.

The stability of the 29-foot portion of the bluff area above the till stratum was approached by determining (1) an estimate of the maximum inclination possible with respect to shear failure, and (2) possible methods of protecting these slopes from frost action, surface runoff, rain impact, and seepage.

The factor of safety of the upper portion of the slope was computed for the existing slopes, using data obtained from field vane shear and laboratory shear tests. Values of 1.1 to 1.7 were obtained for the factor of safety, and since no shear failures were noted, it was concluded that the stability relative to shear was adequate. After consideration of frost action, surface runoff, and seepage forces a blanket of granular material was recommended as being the most effective, economical control of surface erosion. Details concerning these studies are included in a formal report.(2) Recommendations for obtaining stable slopes is shown in Fig. 9.

2. Toe Protection Without Provisions for Stable Slope Development.—Toe protection without provisions for stable slope development constitutes a positive action toward interrupting bluff recession. Under certain economic conditions the postponement of a stable slope above the toe may be the most feasible procedure. Eventually, either nature or further positive action will produce the equilibrium required by the forces which will evolve during the future life of the slope. The delay in obtaining a stable slope will be particularly applicable when (1) there is a shortage of funds for capital investment,

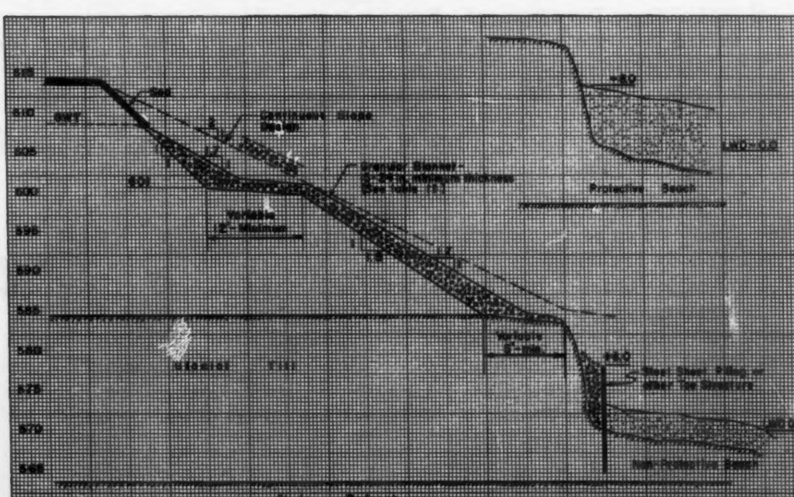


Figure 9. A Corrective Measure for Perry Township Park

and (2) the additional loss at the top of the bluff which might arise because of "inefficient" natural processes is less costly than man-made changes.

Bluff recession will continue after toe protection has been provided, but at a reduced rate, since wave action will no longer be available to transport the detritus from the toe. It is highly probable that equilibrium will be approached gradually as the slopes become flattened by natural processes. Above the toe structure, no drastic changes in the manner in which the bluffs have been receding are envisaged. Past and present profiles reveal a benched slope, similar to the one which was designed.

For Perry Township Park erosion will continue if no direct action is taken but will decrease in magnitude as the slopes become less steep. Frost action will persist regardless of the slope but the velocities of surface runoff will be decreased proportionately, as will the adverse effects of seepage and rain impact. With the existing bluff profile, the minimum inclination of slope, without threatening the stability of the structure (dance hall) near the top of the bluff, is approximately 18 degrees or a 3:1 slope. The value is based upon the assumption that structural stability is endangered when the distance to the edge of the bluff approximates ten feet. If erosion is permitted to continue unabated, positive stable slope development would consist of (1) grading the slope so as to eliminate topographic irregularities which serve to pond water and to act as channels in which runoff is concentrated, and (2) providing a granular blanket for protection from frost action. Seeding or sodding would not prove difficult with the flattened slopes.

Whether temporarily neglecting slope stability above the toe structure is desirable depends largely upon whether (1) the damage incurred by the loss of additional park area exceeds the cost of providing protection against its loss, and (2) the funds are available.

3. Summary.—It is apparent that a decision as to the proper course of action for a given bluff recession problem is unique as is any engineering design problem, and the solution must be obtained on the basis of the variables present so as to represent a balance between the desirable and the attainable. Involved, of course, are: (1) the recognition of the scope of the problem, (2) development of technical equivalents, and (3) economic considerations. Although the four courses of action described in the preceding paragraphs represent varying degrees of endeavor consistent with economic requirements, only two involve design considerations (toe protection with and without provisions for stable slope development). One of the four is not recommended (stable slope development without toe protection) since expenditures are required and the technique will not alleviate bluff recession. The last mentioned procedure (no corrective action) may be followed because of lack of funds, but such a decision is superior to the third course since no expenditures are involved.

SUMMARY AND CONCLUSIONS

Considerable interest has been aroused with respect to the problem presented by the recession of the bluffs adjoining Lake Erie. Although the phenomena were observed in the early days of the missionaries and settlers, it has been only recently that the continuous loss of the valuable natural resource has received much attention. The increasing demand for lake-front areas for both industrial expansion and recreational facilities has accelerated the search for a better understanding.

The immediate challenge facing engineers concerned with providing protective works is the determination of the variables contributing to bluff recession. The rational expression of the characteristics of these variables is also required in a form that can be applied to design criteria. The ultimate objective is to obtain the most economical and adequate protective measures.

Bluff failures are similar to other types of natural slope failures but differ in that additional forces induced by wave and ice action are present. In view of the similarity, a classification of the movements of the slope-forming materials is presented in two broad categories of landslide and erosion. The obvious advantage to such a classification is that the existing knowledge of landslides and of erosion can be incorporated rather than requiring the development of a theory solely applicable to bluff failures.

The ability of a bluff to withstand a given inclination is a function of the stress-strain relationship. Each slope presents a unique set of environmental conditions and variations in the natural deposits. While certain principles will be applicable to all slopes, each bluff must be analyzed in terms of forces and resistance if the attainment of the proper results is to be insured.

A rational approach to the solution of a slope or bluff stability problem may be divided into five normal steps for an engineering study: (1) collection of available data, (2) field investigation, (3) laboratory investigation, (4) analysis of field and laboratory data, and (5) solution. The degree of effort expended on any of the above steps will be a function of: (1) the purpose of the study (research or practical solution), and (2) economic considerations. Of the steps enumerated above, only the last is given major consideration in the paper.

Analyses of bluff stability can be simplified by systematically considering three inter-related problems: (1) beach development, (2) toe protection, and (3) stable slope development. Beach development refers simply to the requirement for a beach for recreational or other purposes. Toe protection is the resistance required at the toe of the bluff to afford resistance to the undermining and erosive action of the lake. Stable slope development is concerned with the area above the level affected by lake activity. By giving sequential consideration to each component a logical series of decisions can be reached. For example, beach development may be either protective or non-protective. In the former the beach is capable of dissipating the wave energy before that energy reaches the toe of the bluff. For the latter, wave energy dissipation may be obtained only on occasions and cannot be relied upon for toe protection. As a result, the selection of the type of beach will have a direct bearing upon the requirements for toe protection. With protective beaches, then, no additional toe protection is required whereas for the non-protective type a structure is necessary. Having fulfilled the requirements for the preceding two components, (beach development and toe protection) the remaining portion of the analysis is concerned with conventional slope design. For unconsolidated materials, some structure may appear economical for maintaining slope stability. If so, the design requirements can be incorporated with those for the toe protection-non-protective beach approach.

For a given bluff recession problem there will be several solutions which are adequate or equivalent from the technical sense. The choice between the alternates will be dictated by economic considerations. The economics must be long range and include maintenance and amortization costs as well as initial investment. Reduction to the standard form of annual cost provides a basis for determining the most economic solution; i.e., the one with the lowest annual cost.

The analysis of the processes operating and contributing to bluff failures indicate that overall stability can be attained only by (1) adequate toe protection from wave and ice action, and (2) stable slope development with respect to landslides and to erosion processes. One of four possible courses of action is generally followed when bluff recession is encountered; namely, (1) stable slope development with provisions for toe protection, (2) toe protection without specific provisions for stable slope development, (3) stable slope development without specific provisions for toe protection, or (4) no corrective action. The first course of action, stable slope development with provisions for toe protection, involves the sequential consideration of the three components (beach provision, toe protection, and stable slope development). The second represents a compromise between (1) the desire to do take positive action to minimize bluff recession, and (2) working within economic limitations. Recession of the bluff will rarely be appreciably alleviated by pursuing either of the two latter courses of action.

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PORT STRUCTURES IN LONG BEACH HARBOR

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SUMMARY

The comparatively rapid growth of the Port of Long Beach is guided by a long range master plan. The purpose of this plan is to provide an efficient and self-sustaining port which will meet the present and future needs of the surrounding area.

The preparation of the master plan has required the development of certain basic planning and design criteria for port structures.

Brief History of Port

Development of the Port of Long Beach began in 1905. In that year private Long Beach interests acquired 800 acres of marshland in the Inner Harbor area and initiated a program of channel dredging and land reclamation. Efforts to establish a municipal port began a short time later and culminated in the opening of Pier 1 in 1911.

An event of even greater significance to the Port's future also occurred in 1911. This was the tideland grant enactment by the State Legislature which conveyed to the City of Long Beach all of the State's rights in the tidelands located within the city boundaries. The grant stipulated that all revenue produced from the tidelands be used for the development of navigation, commerce and fisheries.

Note: Discussion open until May 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2303 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 4, December, 1959.

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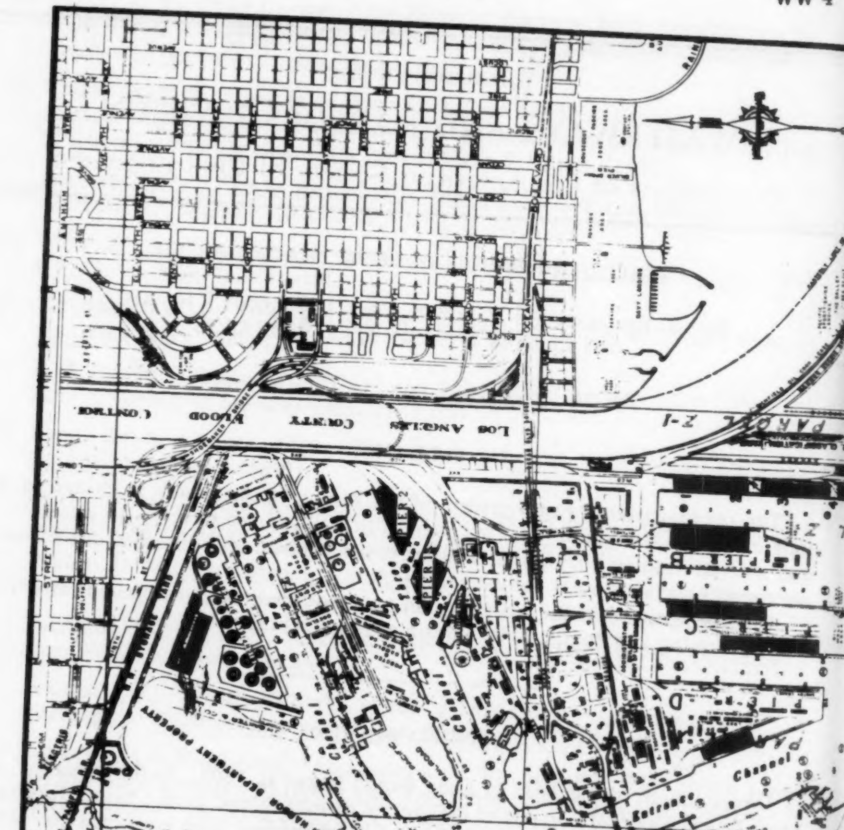

LONG BEACH SHIPYARD
TURNING BASIN
Entrance
Channel
PIER B
LOS ANGELES COUNTY FLOOD CONTROL
CLASSIFICATION YARDS
R R STORAGE YARD
TERMINAL
DIRECT TRUCK ROUTES TO LOS ANGELES INDUSTRIAL DISTRICTS


PORT OF LONG BEACH
CALIFORNIA
0
1000
2000
3000
scale in feet
PIER G
JETTY (Temporary)
PIER F
Fairway
EAST BASIN
WEST BASIN
NAVY MOLE
J.C. 8-5-59

FIGURE 1

During the next 15 years, up to 1926, the Port's commercial growth was steady, though relatively slow. This period saw the accomplishment of several fundamental steps in the development of a major harbor. These included the creation of the Board of Harbor Commissioners in 1917, the dredging of the Inner Harbor and entrance channel to accommodate deep draft vessels, and the correction of a serious silting problem through construction of the Los Angeles River Flood Control Channel. These efforts bore fruit during the ensuing 10 years in the establishment of more than $30,000,000 worth of industrial plants and private marine terminals in the Inner Harbor. Commerce over municipal wharves also increased more rapidly and it became necessary to start the development of an Outer Harbor.

In 1936, oil was discovered in the harbor area, and the first Harbor Department well was brought in 2 years later. It soon became evident that the City of Long Beach would receive substantial revenue which, under the terms of the State grant of 1911, must be used in the development of the Port. It was further realized that to properly guide and coordinate such development, a long range comprehensive plan was essential. The Board of Harbor Commissioners contracted with George F. Nicholson, M. ASCE, and James F. Collins, Consulting Engineers, for the preparation of such a report which was completed in 1940.

The Nicholson-Collins report proposed the development of an East Basin between the Entrance Channel and the Los Angeles River, and a West Basin in the area now occupied by the Long Beach Naval Shipyard. Twenty full-size berths were to be provided in the East Basin and more than 50 berths in the West Basin. The report recommended extensive modern terminal facilities for handling waterborne cargo. Attention was also given to improvement of the street and highway system to give better vehicular access and avoid congestion within the Port.

To a large extent the recommendations of the Nicholson-Collins report were followed in the subsequent development of the East Basin. This development was greatly accelerated by World War II, with its need for increased oil production and additional shipping facilities. The wartime and immediate postwar construction was largely concentrated on Pier A, with the development of Piers B, C and D following in more recent years. An important factor in the planning of these piers was their dual function as oil drilling sites and cargo piers.

The Navy's acquisition of the West Basin area made it necessary to depart from the Nicholson-Collins plan and find other areas for future expansion. At the present time, only one Inner Harbor Port-owned area having protected deep water frontage remains undeveloped. This is the property fronting on Channel 2 in the Inner Harbor, acquired in December 1957, from Pacific Dock and Terminal Company.

Existing Facilities

The Fairway to the Port of Long Beach starts at an 1800-foot wide opening in the Federal Breakwater, 3 miles offshore. This 750-foot wide channel has recently been dredged to a minimum depth of 52 feet in order to allow supertankers to enter the Port. See Fig. 1 for a map of the Port.

Generally, the minimum depth of water in the channel, basins, and slips in the Port is maintained at 40 feet at mean lower low water. The minimum

depth of water at pierhead line is maintained at 35 feet. Consequently, all general cargo ships can enter the Port at all phases of the tide.

The width of slips in the Middle Harbor, which was constructed over 15 years ago, was set at 400 feet. This width enables a cargo ship to navigate the slip with ships tied up at the adjacent side berths. However, in the Outer Harbor, which is now under construction, 500-foot wide slips are being used. These slips are 4 berths long. Some of the reasons for expanding to 500-foot wide slips were that this allowed room for a full berth at the ends, created safer maneuvering room for ships, and the area could be economically obtained.

All of the piers in the Port of Long Beach south of Seaside Boulevard are on filled land. This fill was obtained mainly by dredging in the ocean bottom. However, large quantities of fill have also been obtained from borrow areas outside the Harbor District that were primarily being developed as "cut and fill" trash dumping pits.

The piers in the East Basin were completed in the following order: Pier A, Pier D, Pier C, and Pier B. Pier A, Berths 1-10, for purposes of comparison, can be divided into two parts. The physical features of Berths 1-5 consist of 35-foot wide wharves with tracks, 120-foot wide clear span transit sheds, loading platforms, low level tracks, and minimum back areas for parking and storage. The physical features of Berths 6-10 consist of 50-foot wide wharves with tracks, 200-foot wide clear span transit sheds, loading platforms, low level tracks, and large back areas with warehouses. Records of the annual tons of cargo handled by these 10 general cargo berths, indicate that Berths 6-10 handle approximately twice as much general cargo as Berths 1-5. The layout of the new Outer Harbor, of which Figs. 2, 3, and 4 are examples, resulted from a careful study of the Middle Harbor. Conclusions of this study were, that for maximum operating efficiency, a berth should have a 50-foot wide wharf with tracks, at least a 160-foot wide clear span transit shed (80,000 square feet per berth), a 16-foot wide loading platform, low level tracks, a back area storage space of approximately 50,000 square feet with warehousing as needed, and adequate car parking facilities.

Several types of wharves have been used in the Port development. These are shown on Figs. 5, 6, and 7. Piers B and C were constructed using the wharf section shown on Fig. 5. This style of wharf embodies the use of a steel sheet pile bulkhead with a tieback system, precast concrete piles, and reinforced concrete deck covered with three and one-half feet of fill. A cathodic protection system is used on the steel sheet pile bulkhead. This system has proven satisfactory for initial construction in quiet waters. For construction in areas where the initial construction was subject to more open water conditions, the steel cellular bulkhead shown on Fig. 6 has been used. Each cell when filled with sand becomes a stable element in itself. Concrete caps are added later to finish the section for berthing of ships. This system has been satisfactorily used for the construction of 9 berths. However, one of the chief problems of this type of bulkheading is the lining up of the cells during construction. Experience shows that the alignment may be off as much as 6 feet. This results in a higher cost for the concrete cap. Another type of bulkheading that has been used successfully for 10 berths is a cyclopean concrete quay. Fig. 7 is a typical example. These berths were all constructed in quiet waters. Although this bulkhead is high in first costs, the life of the structure might conceivably be over 100 years. However, the Port usually considers a 50-year obsolescence for structures. An advantage of a concrete

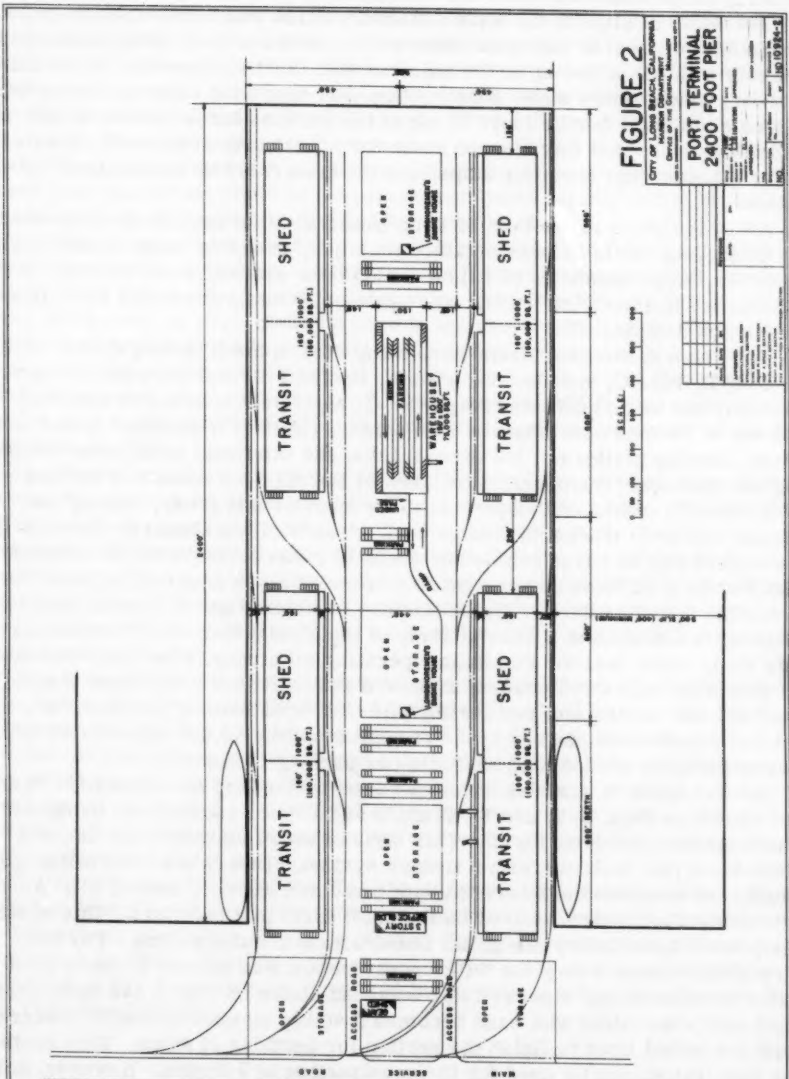


FIGURE 2
CITY OF LONG BEACH, CALIFORNIA
PORT TERMINAL
2400 FOOT PIER
TRANSIT
SHED
TRANSIT
SHED
TRANSIT
SHED
TRANSIT
SHED
OPEN
STORAGE
WAREHOUSE
ACCESS ROAD
SERVICE ROAD
MAIN
2400'
SCALE

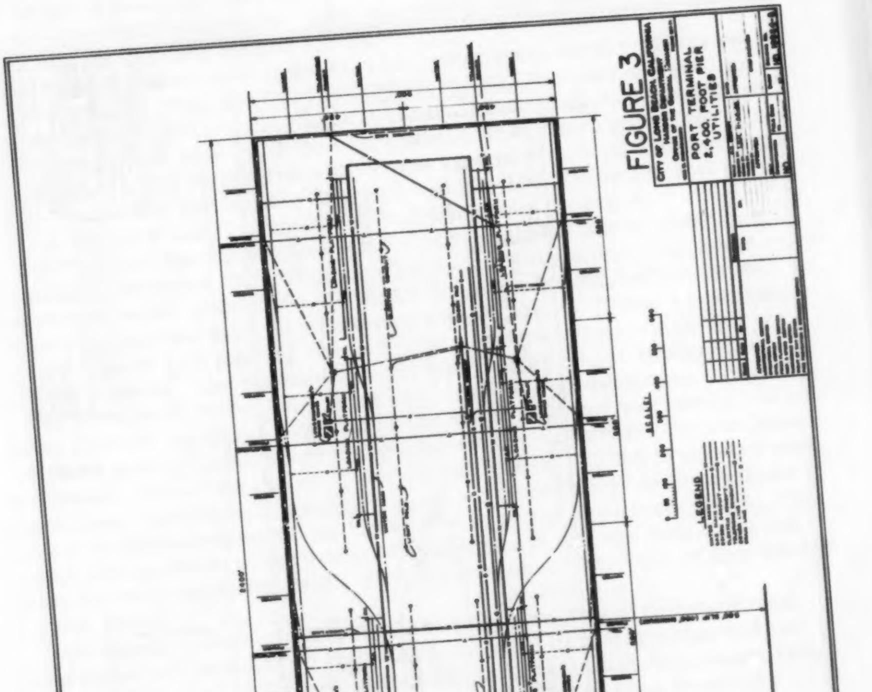


FIGURE 3
CITY OF LONG BEACH CALIFORNIA
PORT TERMINAL
2,400 FOOT PIER
UTILITIES
LEGEND
SCALE

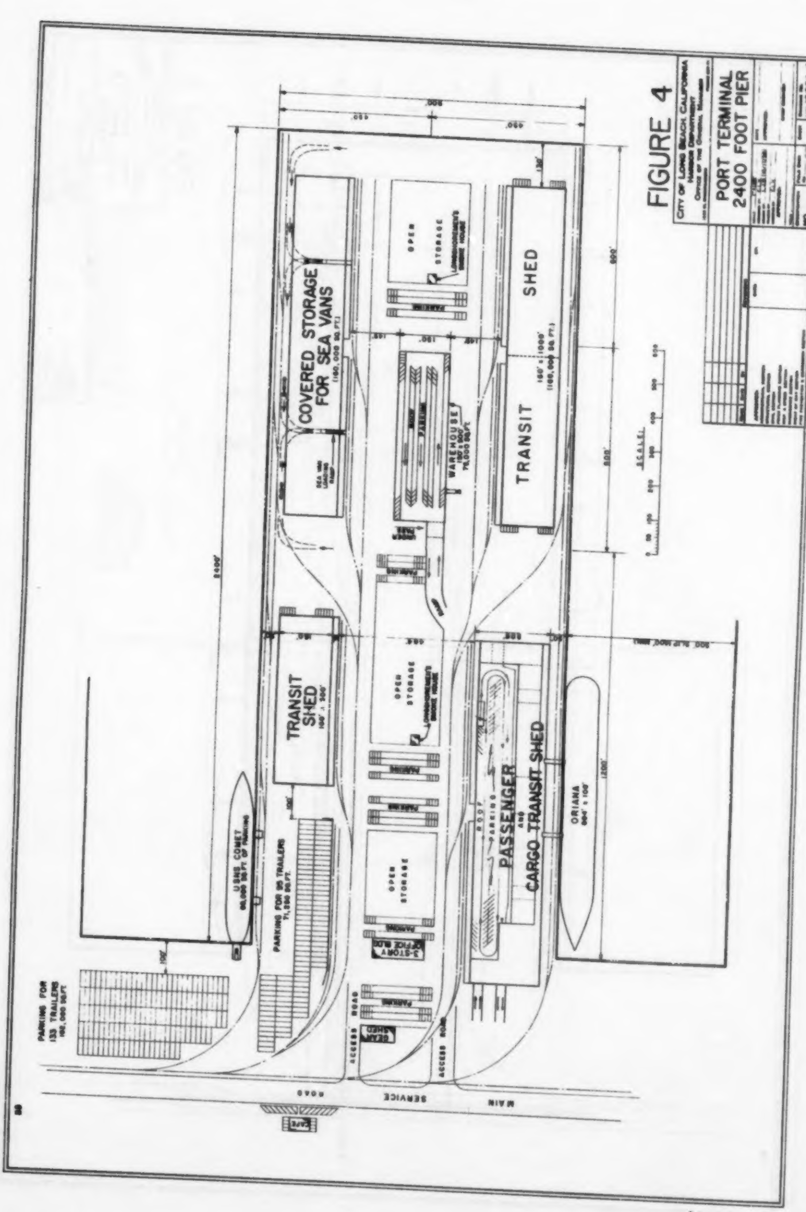


FIGURE 4
CITY OF LONG BEACH, CALIFORNIA
PORT TERMINAL
2400 FOOT PIER
COVERED STORAGE FOR SEA VANS
TRANSIT SHED
WAREHOUSE
OPEN STORAGE
TRANSIT SHED
PASSENGER AND CARGO TRANSIT SHED
ORIANA
USNS COMET
PARKING FOR 95 TRAILERS
PARKING FOR 133 TRAILERS
3-STORY OFFICE BLDG
GEAR SHED
ACCESS ROAD
SERVICE ROAD
MAIN
SCALE

quay is that it requires little steel, and therefore may be constructed during periods of national emergency. Also, because of their mass, they have an inherent strength against unusual forces.

The typical transit shed in the Port is a clear span, steel rigid frame, concrete wall, building that is fully sprinklered and has been designed to meet the requirements of the National Board of Fire Underwriters. These sheds have 18-foot wide by 16-foot high steel rolling doors on the sides, approximately 30 feet on centers. The sheds vary in size from approximately 60,000 square feet per berth at Berths 1 and 2 to 108,000 square feet per berth at Berths 6 and 7. A typical example is shown on Fig. 8.

The Port now has six warehouses and has concluded that in order to meet the needs of shippers, back area warehousing is a necessity. The latest warehouses have areas of over 100,000 square feet and are divided into three rooms with one-foot thick firewalls as shown on Fig. 9. These warehouses are constructed with tapered steel girders, concrete tilt-up walls, and are fully sprinklered. The warehouses cost less than $3.50 per square foot.

The first phase of the Outer Harbor is now under construction. This includes Piers F and G and will furnish the harbor with ten new berths. In planning these piers an analysis of eight different types of wharves and bulkheads was made to determine the most economical design. All the previously used wharf types, as well as several new ones, were considered in the analysis. The cross section shown on Fig. 10 proved to be the most economical. This type is well suited to construction in open water in that the rock dikes and filling can be constructed first. The concrete wharf may then be constructed in quiet waters.

Land transportation to a Port is as necessary as water transportation. Little value will result from fast ship turnaround time if trucking and rail transportation is so bottled-up by inadequate arteries that they cannot keep up with the flow of cargo. For this reason, the Port of Long Beach has

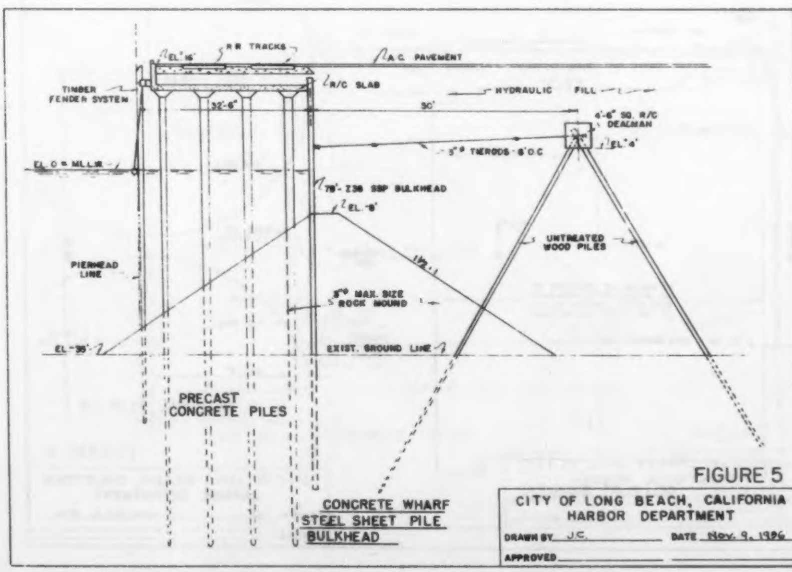


FIGURE 5

recently built three new highway bridges, is extending the local Freeway system into the Port and is revamping the railroad approaches to the Port.

Future Development Trends for Shipside Facilities

Long range planning for the Port of Long Beach is directed toward three main objectives:

1. To accommodate future increases in volume and changes in character of port traffic;
2. To provide for expansion of oil field operations;
3. To make provision for necessary major traffic arteries into and through the Harbor District.

A study of future development trends for shipside facilities should not only look into new equipment and handling procedures, but should also review future growth possibilities of the Port, capacities of existing facilities and economics of operation.

Fig. No. 11 compares the growth rates of population and business activity in Southern California with the increase in general cargo handled by the Port of Long Beach. It can be seen that since 1930 business activity and port commerce have each experienced a six-fold increase while population has been multiplied by two and one-half.

From the evidence of past records, it would be over-conservative to predict future port needs on the basis of population growth alone. This can be seen by the fact that in Los Angeles and Orange Counties since 1940, industrial activity as measured by employment in manufacturing has increased by 270 per cent. This compares to a population increase of 78 per cent in the same period and area.

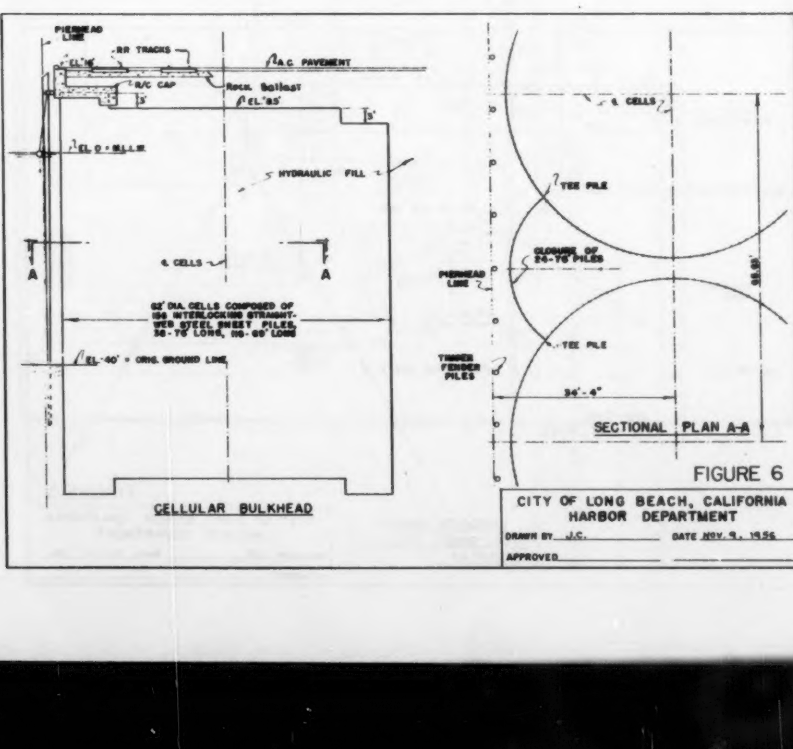


FIGURE 6

There are many factors which point to the continuation of this growth in the future. Among these factors are the continuing westward movement of population and industry throughout the country and the wealth of natural resources in the West remaining to be developed. Another important consideration is the fact that much space remains in the Los Angeles Metropolitan Area for further population and industrial growth, in contrast to the very limited room for expansion in many other port areas.

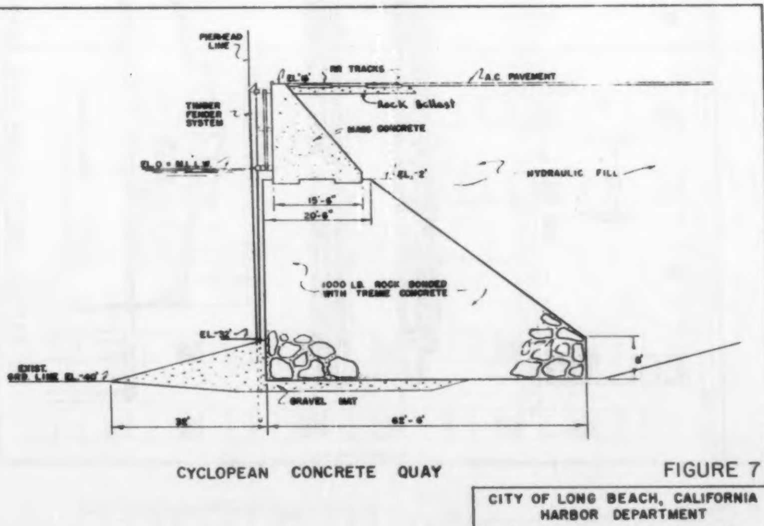
On the other hand, since the Harbor's future growth will be influenced by many unpredictable factors such as world peace conditions, technical improvements in transportation and cargo handling, labor practices, and world markets, a reasonable degree of conservatism is desirable.

In the absence of other supporting evidence in the form of statistical forecasts, the projection of future port commerce is based primarily on the predicted population growth, with conservative adjustment to allow for the past growth relationships of industry and port commerce. This projection indicates a required capacity for general cargo at the Port of Long Beach of 6,000,000 tons per year by 1980. This estimate appears conservative, inasmuch as it represents an average future increase of only 3.5 per cent per year, whereas the past growth rate has averaged nearly 8 per cent per year over the past 30 years.

The second primary factor in estimating future port needs is that of berth efficiency, or the future tonnage capacity per berth. During 1957, 10 general cargo berths handled an average of 94,000 tons per berth.

The cargo loading rate for ships at municipal berths averaged 940 tons per ship per day during 1954 and 1956, as compared to the estimated practical maximum loading rate of the Department of the Army of 540 tons per day.

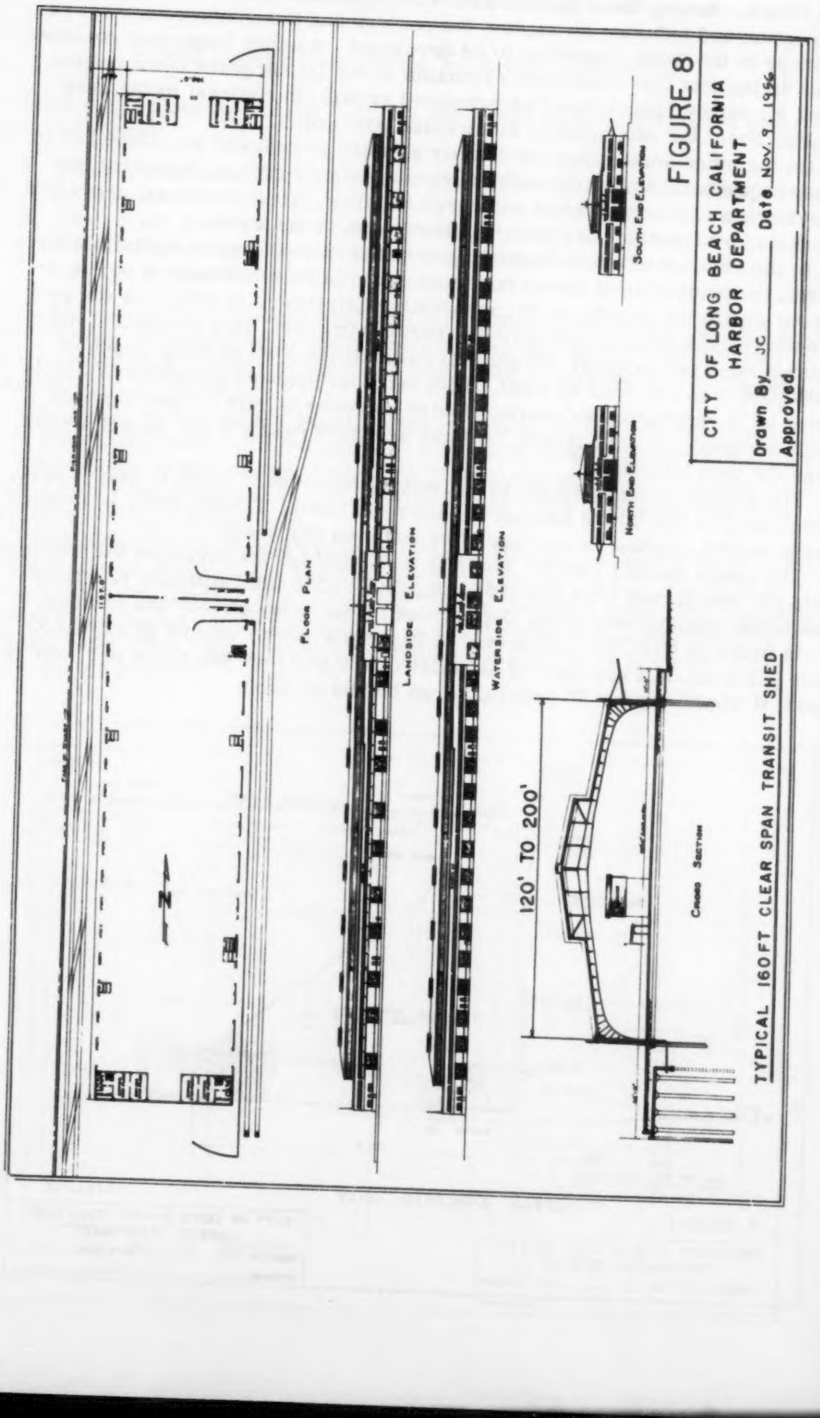
In order to handle the projected 1980 general cargo volume of 6,000,000 tons per year, if a rounded off estimate of 100,000 tons per berth per year is used, it would require 60 general cargo berths by 1980.



CYCLOPEAN CONCRETE QUAY

FIGURE 7

CITY OF LONG BEACH, CALIFORNIA
HARBOR DEPARTMENT
DRAWN BY J.C. DATE NOV. 9, 1956
APPROVED



TYPICAL 160FT CLEAR SPAN TRANSIT SHED

In addition to its general cargo capacity, the future harbor must also make adequate provision for other types of cargo and for a wide variety of port activities. Included among many others are additional bulk terminals with storage elevators, additional bulk oil terminals, fuel docks, passenger terminals, seaplane facilities, explosives terminals, berths for marine contractors and repair facilities. While future requirements for these facilities cannot be projected in detail, it is estimated that they will require at least one-fourth the total number of berths, or say 20 additional berths.

It would appear from this that the Port of Long Beach will need about 80 berths total by 1980. There is a possible development of about 40 berths, in the Inner and Middle Harbors. This leaves about 40 berths to be developed in the Outer Harbor.

As a means of guiding current planning effort toward these long range objectives, a series of Master Plan maps was prepared. These show respectively 5-year, 10-year and 20-year phases of future development in addition to existing features. The maps show the general outline of future piers and indicate possible land uses.

These plans were prepared, using the philosophy that the Master Plan be fixed in basic concept only and that the details be flexible. These plans show no details of the type structure which might be built on the various land masses; instead, just the land masses are shown with target dates for their construction.

The planning represented on these maps is in accord with the Preliminary Master Plan for the City of Long Beach. It is also coordinated with the City-wide traffic planning conducted by the City Engineer's office.

Also, if any of the proposed new methods of cargo handling, or perhaps some which have not as yet been visualized, should come into being, the amount of cargo per year over a berth may be greatly increased, thereby reducing the number of berths needed over any given span of years. Because of

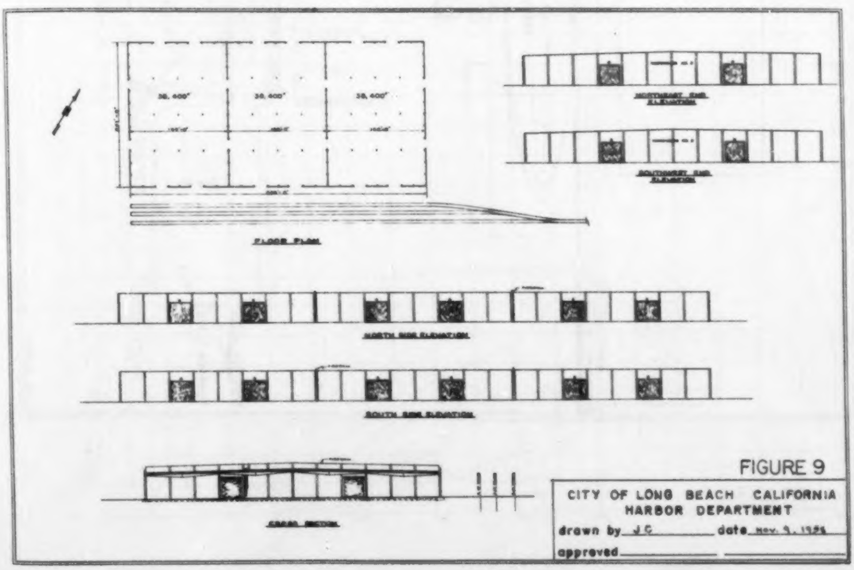


FIGURE 9

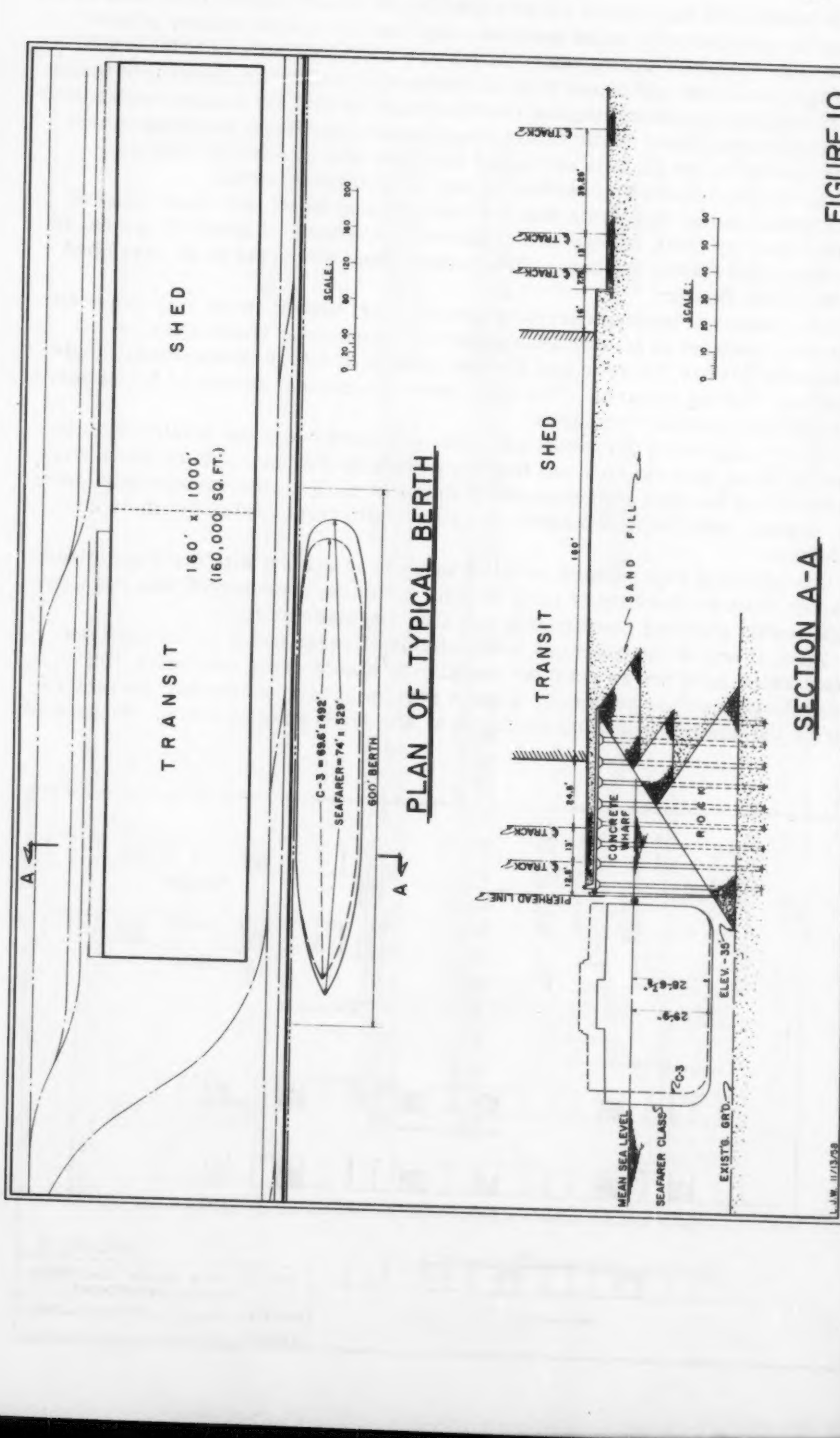

PLAN OF TYPICAL BERTH
TRANSIT SHED
160' x 1000'
(160,000 SQ. FT.)
C-3 = 69.6' x 492'
SEAFARER = 74' x 529'
600' BERTH
SCALE:
SECTION A-A
TRANSIT SHED
SAND FILL
CONCRETE WHARF
PIERHEAD LINE
MEAN SEA LEVEL
SEAFARER CLASS
EXIST'G. GRD.
ELEV. -35'

FIGURE 10

this, the proposed Master Plan is made up of increments which are complete entities in themselves and need be built only as they are needed. The basin surrounded by Pier F and Pier G is such an example.

The development program represented in the Master Plan maps will make possible the addition of 10 berths by 1963, 9 more by 1968, and 20 additional berths by 1978.

The basic land areas of these plans, as shown on Fig. 1, are called Piers F, G, H and J. They are designed such that they may be used as open storage areas, traditional transit sheds for break-bulk cargo, combination break-bulk cargo and passenger sheds, roll-on, roll-off facilities, or lift-on, lift-off facilities. Fig. No. 4 indicates how these various types of facilities may be used in any combination upon a typical pier such as Pier G or Pier H, all depending upon how the methods of cargo handling develop.

Economics of Typical General Cargo Berth

A general cargo berth, as the name implies, may be used for the importation or exportation of a great variety of cargo. A general cargo list would include canned and frozen foods, cotton, lumber, foreign cars, cloth, soaps, animal hides, newsprint, structural steel and tools. In fact, practically any article produced or used in the United States, except for bulk and liquid products, may be considered as possible general cargo. Because of this wide variety of products, general cargo necessarily comes in various shapes. However, some shipping companies are packaging smaller packages in larger group units by using pallets or "walk-in" containers.

Presently, a general cargo ship is loaded or unloaded out of from one to five of its hatches by ships' gear at an average rate, estimated by various authors and agencies of from six to twelve tons per hour per hatch. These are not short period rates, but rates which consider all down time for any reason. At the Port of Long Beach, two 8-hour shifts are ordinarily worked

GROWTH OF LONG BEACH HARBOR

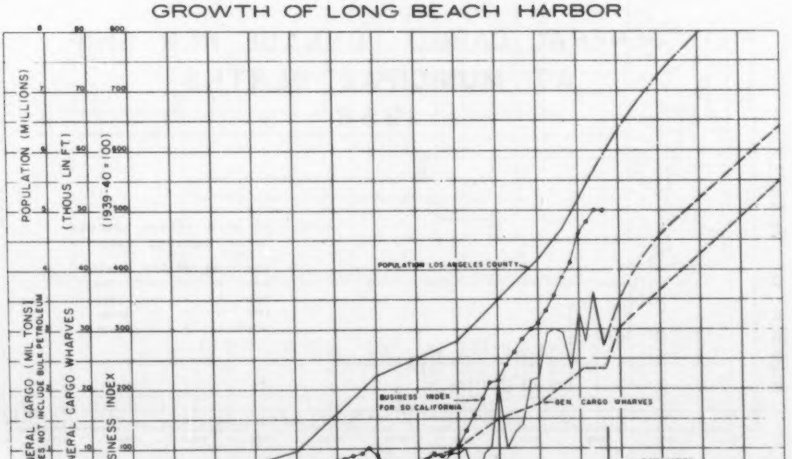


FIGURE 11

each day. If all five hatches were worked, this would amount to a rate of from 480 tons to 960 tons per day. The average rate in 1956 at all municipal general cargo berths in Long Beach was 940 tons per day. The average general cargo ship that docks in the Port of Long Beach loads and unloads a total of 1,880 tons over a 44-hour stay. This 44-hour period is considered to include four 8-hour working shifts.

However, considering 10 berths in 1954 that were handling general cargo but with no steel, foreign cars or lumber, the rates ranged from 397 tons to 1,346 tons per day with an average rate of 771 tons per day. Also on four of these berths where the transit sheds had areas of 80,000 square feet or over, and where the distance on the wharf from pierhead to the face of the transit shed was 50 feet, the average rates exceeded 1,000 tons per day.

An average of 62 ships were accommodated at each of these ten berths in the fiscal year 1956-57, and loaded or unloaded an average of 95,000 tons per berth for a total average annual revenue per berth of $76,000.

Fifty-two per cent of the general cargo shipped through the Port of Long Beach is foreign, while 24 per cent is coastwise, and 24 per cent is intercoastal. Of this cargo, approximately one-half is inbound and one-half is outbound.

Charts shown on Figs. No. 12 and No. 13 indicate tons of cargo per ship and hours in port per ship.

The annual revenue for a general cargo berth is derived from pilotage, dockage, wharfage, storage and demurrage, utility and rental charges. A typical example would be the docking of a C-3 ship. The Harbor Department shares in the pilotage of $0.012 per gross ton for docking the ship and also in the same amount amount for taking the ship back out to sea. While the ship is at the dock, the charges for typing up are $70.50 per 24-hour day. For every foreign ton of cargo that crosses the pierhead line, the wharfage charges are $0.80 per short ton. These same charges are $0.40 per short ton for

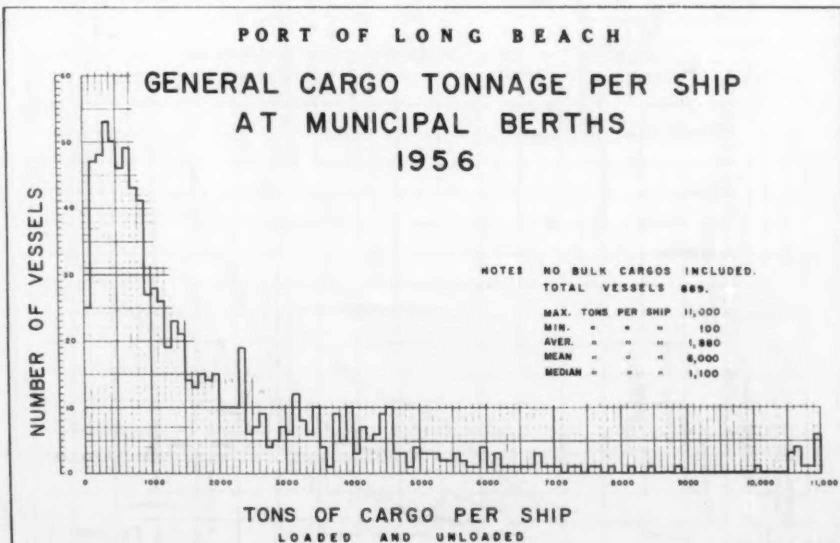


FIGURE 12

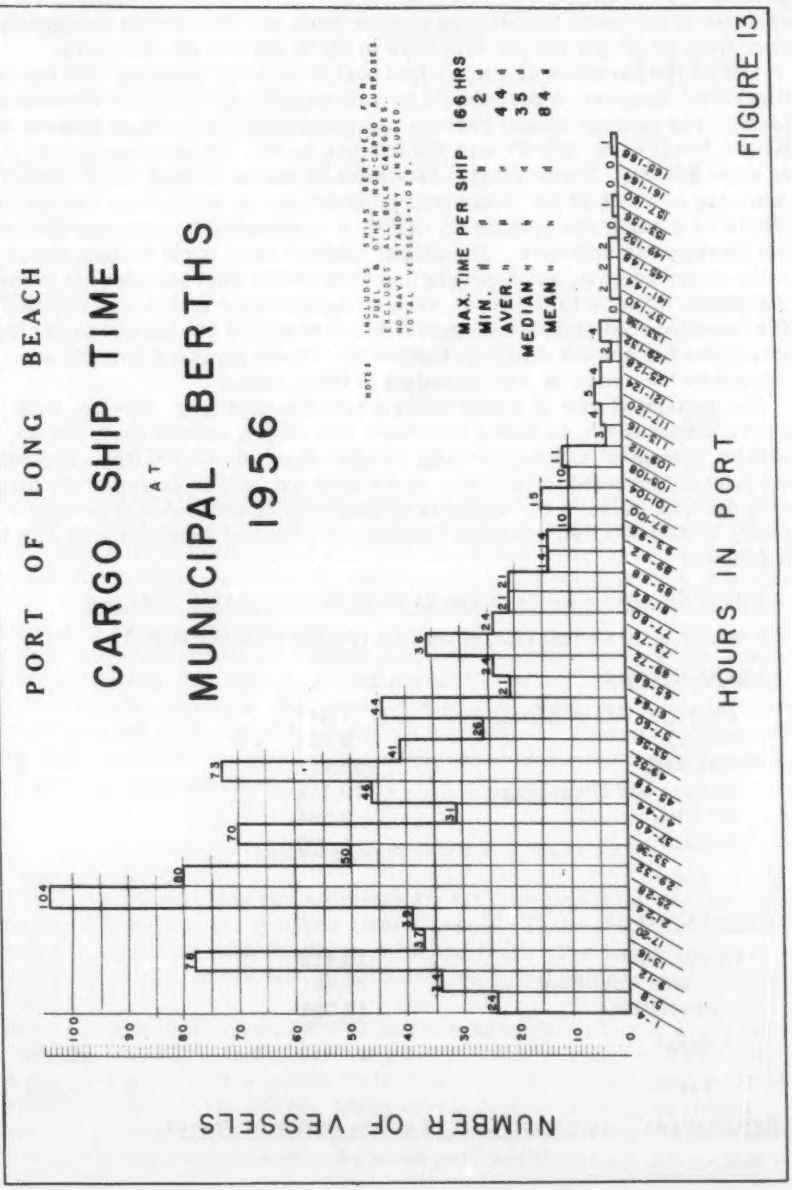

PORT OF LONG BEACH
CARGO SHIP TIME
AT
MUNICIPAL BERTHS
1956
NOTES: INCLUDES SHIPS BERTHED FOR FUEL & OTHER NON-CARGO PURPOSES.
EXCLUDES ALL BULK CARGOES.
NO NAVY STAND-BY INCLUDED.
TOTAL VESSELS-1028.
MAX. TIME PER SHIP 166 HRS
MIN. 2
AVER. 44
MEDIAN 35
MEAN 84
HOURS IN PORT
NUMBER OF VESSELS

FIGURE 13

coastwise cargo. The shipper has between five and ten days free time, depending upon destination or origin of cargo. If, after this free time, the cargo remains at the berth facility, the shipper must pay demurrage charges that vary from $0.35 per ton per five days to $0.70 per ton per five days.

One of the purposes of a municipal port is to bring business into the area it serves. However, a port should be self-supporting and make a reasonable profit. The average annual revenue per general cargo berth as already shown for the fiscal year 1956-57 was $76,000 per berth. The average annual charges were $66,000. These actual costs were based on general cargo berths operating at about 50 per cent capacity which seems to be about the greatest efficiency a berth can operate at, unless it is designed for a particular cargo and therefore is selective. The annual charges for a berth include direct costs, prorated cost, and depreciation. The direct cost includes all items chargeable directly to the berth, such as maintenance and port personnel. The prorated cost includes all port items that should be charged to the berth operations but are not directly chargeable. These prorated charges are charged to the active berths according to their usage.

The estimated cost of constructing a new 600-foot long, 450-foot wide general cargo berth, including the dikes, fill, wharf, transit shed, tracks, utilities, pavement and engineering, is approximately $2,000,000. The rock and sand fill included in the above price does not have to be replaced; therefore, the total value of the depreciable facilities is estimated to be approximately $1,600,000. An economic analysis of a typical general cargo berth is as follows:

Actual Revenue - 1956-57 Fiscal Year (Based on 1956 Charges)

Average of 10 General Cargo Berths (95,000 tons per berth)

Annual Revenue		
Pilotage (20% of Total)	$ 1,457	
Dockage	6,881	
Wharfage	53,766	
Storage and Demurrage	10,114	
Utilities	2,584	
Rentals	1,409	
Total		$ 76,200
Annual Costs		
Direct Cost	10,112	
Prorated Port Cost	37,481	
Depreciation	18,755	
Total		66,348
Profit		$ 9,852

Hypothetical Revenue and Cost of a New Optimum Facility

Construction Cost	
Earth and Rock	$373,700
Concrete Wharf (Prestressed Piles)	517,200
Transit Shed and Utilities	762,000

Railroad Tracks	$ 83,200	
Pavement	49,900	
Engineering	178,600	
Total		$1,964,600
Less Earth and Rock		373,700
Total of Depreciable Facilities		$1,590,900

Annual Revenue (Based on 1959 Charges)

Use 1,000 tons per day x 252 days per year	
50% use = 126,000 tons	
Revenue	$ 115,000

Annual Cost

Direct Cost	$ 19,000	
Prorated Port Cost	44,000	
Depreciation	19,000	
Total		82,000
Profit		$ 33,000

The approach to the depreciation rate on a shipside terminal facility seems to be somewhat a matter of opinion. Some believe that the rate should be a straight line depreciation, based on 75 years; others believe that it should be a straight line, based on 50 years obsolescence. Another approach is a sinking fund based on government bond interest rates of approximately 2 per cent and based on 50-year obsolescence.

As the wharf structure and transit sheds are built of concrete and steel and are adequately maintained, the structural life of the facilities may well exceed 50 years. However, the obsolescence of the structure from the viewpoint of usability probably will not exceed this period. Therefore, the middle of the road sinking fund approach was used, based on 50-year obsolescence and 2 per cent interest rates.

Future Development Trends for a Typical Pier

The general cargo ship replacements for the C series ships are the seafarer, the clipper and the freedom class of ships. These ships vary little from the C series ships in length, breadth and draft. The largest of these ships, the seafarer, is 529 feet long, 75 feet wide, and has a loaded draft of 30 feet.

Ships for specialized use, such as tankers, are getting larger every year. A recent tanker ship's design was for a 103,000 ton ship that was 900 feet long, 135 feet wide, with a loaded draft of 48 feet. Bulk cargo ships may also approach this size in the future. Other specialized uses, such as trailer ships, will probably be the same general size as general cargo ships since they will be using approximately the same port facilities.

New Type Cargo Ships

At the present time plans for many new types of cargo ships are being made. The basis of all of the proposed new designs is to reduce ship's port

time by revising cargo handling methods such as to reduce manual labor to a minimum and thus greatly reduce the cost of loading and unloading cargo. Although all of these proposed plans and designs are in a formative stage, it is essential that all new plans for port terminals provide facilities which are versatile enough to fit in with any of these proposed ship designs and be so designed as to be able to move cargo to or from shipside at the fast loading and unloading rates which are universal in the proposed designs of all of these ships.

The following table gives some idea of the difference in short period loading rates between present break-bulk type cargo ships and the envisioned ships.

	Rate Tons per Hour
Break-Bulk Cargo Ship	150 to 400
Pallet Ship (Sideport-Fork Truck)	350 to 700
Container Ship (Lift-on, Lift-off)	1200 to 3000
Trailer Ship (Roll-on, Roll-off)	6000

As can be seen from the above chart, future rates of loading and unloading ships could be from two to fifteen times as fast as present loading and unloading rates.

Fig. No. 2 (Typical general cargo covered berths), Fig. No. 3 (Typical open area berths), and Fig. No. 4 (Four possible uses of a typical berth) indicate several possible ways of using a typical 450-foot wide by 600-foot long (6.2 acres) berth area. These uses are:

1. An open area for handling cargoes such as lumber, foreign cars or sea-vans.
2. Open area for "Roll-on, Roll-off".
3. Covered storage for sea-vans.
4. Passenger terminal.
5. Transit shed covered area for general cargo.

Open Area

The open area provided at the berth provides two wharf tracks, and two low line tracks for rail transportation. Also provided is an adequate truck loading retaining wall, utilities, lighting and longshoremen facilities.

"Roll-on, Roll-off"

"Roll-on, Roll-off" cargo handling describes a containerized shipping method which allows loaded trucks to drive directly from the pier into the hold of the ship and either leave their trailers to be transported to the desired port or be transported as a complete unit (trailer and truck). This type of cargo handling requires the use of special ships which have loading ramps on both sides and at the stern and have been internally designed to accommodate the trucks with their trailers.

As shown on Fig. No. 4 depicting the terminal layout for a shipment of "Roll-on, Roll-off" cargo, the emphasis is on parking space for the trailers. Enough pier space has to be allotted to accommodate both the inbound and outbound shipment.

Sea-Vans

Reusable containers that are used to transport products from the producer to the consumer via ships are known as "Sea-Vans." These containers are generally constructed of aluminum or steel, and are equipped with handling rings. The containers may be equipped with refrigeration. Their man advantages are that they reduce handling cost, breakage and pilferage.

Container capacity usually varies from 10 to 15 tons in a range of sizes of approximately 8 feet wide, 8 feet high, and 8 to 35 feet long. Cranes used to load or unload these containers may be either shore based or ship-mounted. Shore based units can only be economically justified when there is heavy service between a few ports. Otherwise, ship-mounted cranes will be advantageous.

New container ships are more generally being equipped with ship-mounted cranes. These cranes are of the traveling gantry type with extensible jibs for outreach over the dock, and have a design capacity of 25 tons with a four to five minute cycle. Each crane has a capacity of 350 to 650 measurement tons per hour.

Shipside facilities for handling sea-vans consist of fork lift trucks and straddle trucks, each capable of handling the individual containers. Open sided transit sheds, wide apron wharves, and special loading ramps are also desirable. (See Fig. No. 4).

Passenger Terminal

A passenger-cargo transit building, as visualized, should be a three-level dual operational facility, efficiently and comfortably handling both passenger and cargo traffic. The lower level or ground floor is entirely for cargo movement and storage, and in general will be a typical transit shed plan. The second level will be the basic approach, circulation and parking facility for vehicular transportation of tourists, visitors and luggage for passenger liners. Vehicles enter and leave this second level by means of an elevated access roadway ramping from the main thoroughfare. Thus, cargo handling facilities of the ground floor will be completely separated from the passenger transportation traffic. Additionally, the second level provides the efficiency and convenience of immediately adjacent driveway and parking facilities for passenger-and-luggage transit operations. Vehicular traffic areas should be ventilated by decorative grilles between columns or otherwise at the exterior walls.

The second level will also contain a large screened area allocated for luggage transfer and inspection by U. S. Customs. The area can be immediately adjacent to a long passenger-and-luggage traffic island, thus providing an efficient and convenient baggage transfer system. Office accommodations are also desirable at one end of this second floor.

Vehicular ramps and escalators should lead from the second floor upwards to a third level providing comfortable transition for passengers and visitors. The third level should be nearly level with the Promenade or "A" deck of passenger vessels for comfortable gangway access. The landside longitudinal half of the third level could provide additional driveway and parking facilities. Touring busses and taxis can stop at an extended loading island, providing convenient access accommodations for transient passengers of incoming vessels. The waterside longitudinal area should offer a sight-seeing promenade balcony the full length of the building and various reception salons,

foyers, waiting rooms, escalators, and accommodations arranged to provide the maximum convenience for passengers and visitors. A large restaurant may be incorporated at one end of the upper level, offering a spectacular high-level view of harbor activities through window-walls. Office areas can also be provided at the ends of the third level.

In brief summation, the contemplated passenger and cargo transit building superimposes passenger vessel accommodations above a typical transit shed floor plan, combining but separating these two different operations. In addition to isolating the activities, the superposition scheme raises the floor level of the passenger facilities to similar upper deck heights of the passenger ships for horizontal gangway access.

General Cargo Berth

Using the C-3 as typical of the American ships in use today, and the seafarer class ship as typical of those in future use, minimum berthing standards can be established for a general cargo berth. The ship berth should be 600 feet in length and should have a depth alongside of 35 feet at mean lower low water. The minimum distance between piers should be 400 feet to allow for the ships at either pier, bunkering and ship movement in between. (See Figs. No. 2 and No. 10).

Alongside the berth there should be a covered transit storage area. The Port of New York Authority, using a typical general ship cargo of 12,500 measurement tons half discharged and half loaded, and assuming 70 cubic feet per measurement (revenue) ton, 14-foot stack height, 25 per cent waste space, and 50 per cent working aisles, estimates the need of 90,000 square feet for this area. Mr. Kenneth Peel of the U. S. Army Corps of Engineers, in his discussion on transit shed design, recommended 100,000 square feet per berth for transit storage. His figure was based on a ship with 770,000 cubic feet of cargo space, 12-foot stack height, and 40 per cent wasted space.

Using the actual Port of Long Beach figures of approximately 2,000 tons per ship, two days per ship, maximum free time, 20-foot stack height, and 40 per cent wasted transit shed space, the area of transit shed needed to keep a ship at the berth 100 per cent of the time is estimated at 93,300 square feet.

From the above rationalizations, it would appear that a logical transit shed area should therefore be at least approximately 90,000 square feet. However, experience has shown that transit sheds containing 80,000 or more square feet have been adequate for cargo unloading rates of 1,000 tons per day and more. As shown on Figs. No. 2 and No. 10, the transit sheds have 80,000 square feet per berth (160 feet wide and 500 feet long).

In order to handle large units of cargo and the resulting equipment, the transit sheds should have a minimum eave height of 20 feet, and the entire waterside of the shed, except for columns, should open. The minimum door height should be 18 feet. The use of as few interior columns as possible helps the flow of goods.

All ramps, transit shed openings, wharf load limits, and back area layouts should be designed to facilitate present and future materials handling techniques and equipment.

The entire pier area should be paved. Asphalt pavement has proven satisfactory for both the open and covered areas.

Adequate lighting should be provided for 24-hour use of the entire pier. Also, in order to provide low fire insurance, fire resistant construction and

fire protection equipment such as automatic sprinklers in buildings, fire hydrants, and fireboats should be provided.

The transit shed should be set back from the pierhead line about 50 feet in order to allow a wharf space for loading and unloading cargo. This 50-foot width allows for the installation of as many as three wharf tracks or two wharf tracks and a crane rail. However, for general cargo purposes, the use of one or two wharf tracks is sufficient. The wharf should be designed for a minimum surcharge of 750 pounds per square foot. This loading allows for full flexibility in the use of the wharf area for cranes, trucks and trains.

In order to adequately hold the ship to the berth, bollards and cleats should be alternately spaced along the pierhead line about 65 feet to 75 feet apart. To supply the utility requirements of the ship, two water outlets two inches in diameter protected with back flow preventors, one combination 208-120 volt power outlet, and one telephone outlet should also be provided at each berth.

It is estimated that over two-thirds of the land transportation to a general cargo berth is furnished by trucks. A marginal type wharf where a loading platform extends the full length of transit shed will provide approximately 40 truck stalls.

The average truck load is ten measurement (revenue) tons. It takes about two hours on the average to load or unload a truck. On the basis of an 8-hour day, 40 truck stalls will handle 160 trucks per day (1600 measurement tons of cargo). Other trucks could also be loaded or unloaded by the use of fork lifts.

Rail transportation to and at the berth, though decreasing in volume, still is essential. Bulk commodities and heavy loads are still transported almost exclusively by rail. Also, in times of national emergency, rail transportation gains added importance. Trackage needed at a general cargo berth includes one through track and at least one side track on the wharf, and at least one track at the loading platform.

Back area tracks should provide adequate loops for efficient train movements, and adequate storage.

Back area storage space, either covered or uncovered, should be provided for each berth. This space should roughly equal one-half the transit shed space (40,000 square feet). Lumber, cotton, structural steel, foreign cars, and sea-vans are examples of cargo requiring this back area. Some of these cargoes, such as cotton, require the use of warehouses or covered storage.

The pier should also provide adequate space for parking for longshoremen's cars, offices for shippers, terminal operator facilities and restaurants. Parking facilities for approximately 125 cars per berth should be provided. If space limits are critical, multiple story parking garages could be built. Each shipper and terminal operator usually requires office space at the berth. Terminal operation (stevedoring) facilities should be built on the approximate basis of one facility for five berths. A few coffee shop type restaurants should be provided in the entire harbor area.

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A CASE OF CRITICAL SURGING OF A MOORED SHIP[a]

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ABSTRACT

An analysis is made of the circumstances in which an oil tanker, berthed at a solid quay wall in a rectangular basin of a port subject to influences of long period waves, developed critical surging motions and broke adrift after rupture of its mooring lines. The response characteristics of the ship are examined in the light of the non-linear equation of motion for longitudinal surging and the best available information on the conditions of mooring, the state of the sea and the time of the tide. From the location of the ship in the dock it is inferred that the possible modes of oscillation of the water body in the harbor basin that could have influenced the ship would have been the uninodal, biodal, quinquinodal (and possibly sextinodal) seiches. It is then shown that with the rise of tide in the dock and the accompanying known and probable amplitude increases of the various modes of seiches, only the quinquinodal seiche with a period of 22 seconds could have been responsible for the development of rope tensions sufficient to fracture the mooring lines.

INTRODUCTION

In harbors that are exposed to large tracts of ocean along lines of approach of cyclonic and frontal storms, ingress of long period waves, originated with such storms, sometimes excites oscillations of the water masses (seiches) in the basins and gives rise to troublesome motions of moored ships. (cf. Wilson(1) and papers of Communication 1, Section II of the XIXth International Navigation Congress, London, 1957).

The behavior of a ship at a berth is often closely linked with its location in the dock with respect to the nodes of the water oscillations, with the state of

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a. Contribution from the Dept. of Oceanography and Meteorology, Agricultural and Mechanical College of Texas, Oceanography and Meteorology.

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the tide and the manner of the ship's mooring (tightness or slackness, number and elasticity of its mooring lines), with its size and draft (light or loaded), and of course, with the magnitude of whatever seiches may be prevalent during occupancy of the berth. The combination of these factors sometimes produces unexpected results in that a ship in an unfavorable position may exhibit critical surging, sway or yawing motions in relatively mild sea conditions (to which neighboring ships might be almost immune), whereas another ship in the same berth on a different occasion may lie relatively quiet and unaffected in considerably greater sea disturbances. The peculiarities of such occurrences led the writer[2] to attempt an analysis of the idealized motions of a moored ship located at the node of a seiche. The emphasis was on longitudinal surging which has since been further considered theoretically by Abramson and the writer[3] and successively by Russell,[4] Joosting,[5] O'Brien and Kuchenreuther[6,7] and Wilson[8,9] again. In the course of these presentations several errors in the original formulation of the problem have been corrected and the theory now seems capable of explaining the apparently enigmatic behavior-patterns of ships referred to above. It is the purpose of the present paper to use this theory in explanation of the circumstances in which a tanker of 18,000 dead-weight tons broke adrift from its moorings in a harbor basin.

Case History

The tanker, of overall length 480 ft., beam a = 66.1 ft. and mean loaded draft D = 27.87 ft., entered port and docked at low tide in the early hours of May 5, 1952, along the solid vertical quay wall at one end of a long rectangular basin. The ship was breasted off from the fixed timber waling on the quay by conventional timber floating fenders and tied by the system of mooring lines shown in Fig. 1. The location of the ship in the 2100 ft. width of dock (Fig. 2) was such that its mid-ship distance b from the long side of the dock, nearest its bow, was 590 ft.

The rectangular basin was dimensionally susceptible to several possible modes of transverse oscillation, shown in Fig. 2. The existence of such seiches on occasion in the port concerned has been established and their natural periods τ can be computed with a fair degree of accuracy from the equation.[10]

$$\tau = \frac{2B}{m\sqrt{gd}} \tag{1}$$

wherein B is the width of the basin, m an integer (1, 2, 3 . . .) defining the harmonic mode, g the acceleration due to gravity and d the uniform water

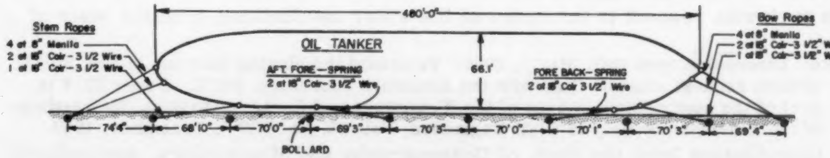


Fig. 1. System of mooring-lines used for securing oil-tanker at berth

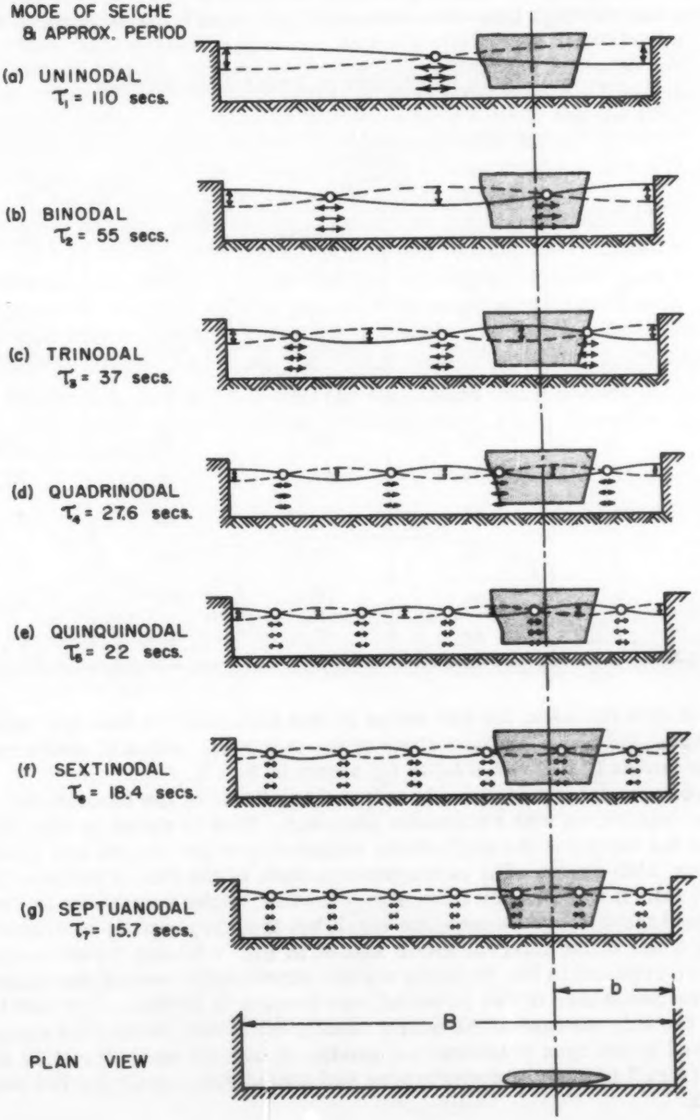


Fig. 2: Location of oil-tanker in dock in relation to seven harmonic modes of transverse oscillation of the water body

depth. For the water depths prevalent at the time of docking of the ship and 6 hours later at high tide, the values of τ are found to be as given in Table I.

Table I. Periods of Transverse Seiches

Harmonic Mode m	On Docking (Low Tide)		High Tide	
	Water depth d (ft)	Seiche Period τ (secs)	Water depth d (ft)	Seiche Period τ (secs)
1	44	111.5	46	109.1
2	44	55.7	46	54.5
3	44	37.2	46	36.4
4	44	27.9	46	27.3
5	44	22.3	46	21.8
6	44	18.6	46	18.2
7	44	15.9	46	15.6

Period differences at the two states of tide are minor so that it is possible to represent the nodal configurations of the individual modes of water oscillation by the series of end views (a) to (g) shown in Fig. 2.

From available tide records only the amplitude of the fundamental uninodal seiche was known with reasonable accuracy. This is shown in Fig. 7(b) and forms the basis for the conjectural magnitudes of the binodal and quinquinodal seiches, also shown. The corresponding state of the tide is reflected in Fig. 7 (a). It will be seen from Fig. 7 (b) that seiche amplitudes at the time that the tanker docked were quite small but that they rapidly increased on the rising tide. In the interval shown shaded in Fig. 7 (a) and (b) the surging of the ship resulted in her breaking adrift. Attempts to remoor her failed even with the assistance of two powerful tugs brought in between 1100 and 1130 to force the ship abreast of the quay. Finally the tanker was pulled clear and removed to the open roadstead for safety. It was not until almost 18 hours later (Fig. 7 b) that the disturbances had abated sufficiently for the ship to redock.

Dimensions of the Ship Mooring Lines

In order to estimate the behavior of the ship in surging it is necessary to make some initial assumption in regard to the lengths S_0, Fig. 3 (b), which the individual mooring lines would have had the transient moment of the ship's passing through the equilibrium position it would occupy in the absence of any

disturbance. This entails some consideration of mooring line geometry. In the equilibrium position, A, Fig. 3 (a) and (b), the horizontal projection of a typical bow-rope, AB, of length S_0, is L_0, its horizontal angle of inclination to the dock front, θ, and its vertical projection above the quay level, H. In displacing the longitudinal distance X along the quay from A to A' (Fig. 3a), the horizontal projection of the rope is increased by d L to L and the corresponding length of the rope, allowing for extension under stress, from S_0 to S. It is convenient tentatively to regard the rope situations AB and A'B as disposed in the same vertical plane, as shown in Fig. 3 (b).

The dimension S_0 will obviously be greater than the direct length A'B A'B $(=\sqrt{L_0^2 + H^2})$. Since the tanker was moored during a period of slight surging at low tide it is almost certain that the berthing crew would have attempted to make the ropes as tight as possible by taking advantage of each lunge of the ship forward or aft. Hence the probability is that the dimension S_0 can be taken to exceed $\sqrt{L_0^2 + H^2}$ by only a small amount, ΔS_0, estimated at 1 ft. for locations 1, 2, 7 and 8, 0.8 ft. for locations 3 and 6 and 0.6 ft. for locations 4 and 5 (Table II). For all the ropes shown in Fig. 1 therefore S_0 has been determined from

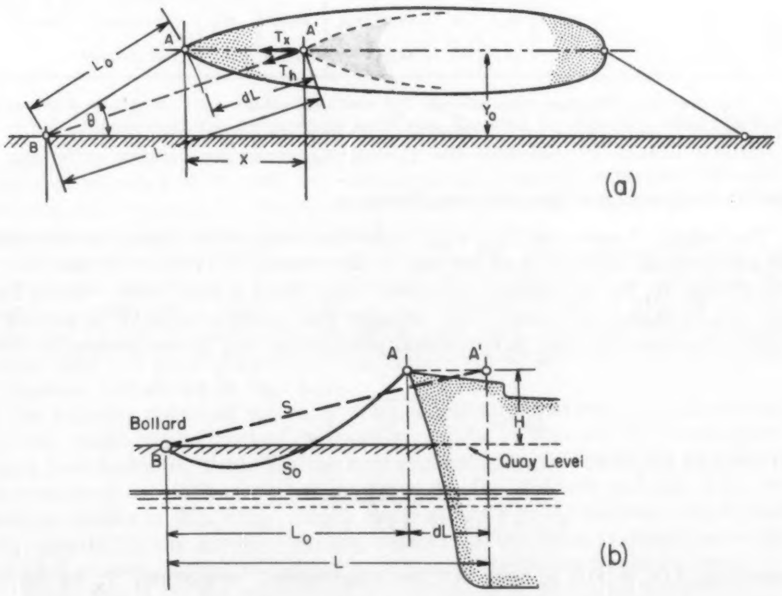


Fig. 3: Mooring line geometry: (a) plan view of ship showing sternward surge translation X; (b) side elevation view of ship projected in the vertical plane of the ropes

$$S_o = \Delta S_o + \sqrt{L_o^2 + H^2} \tag{2}$$

for the values of ΔS_o, L_o and H, estimated from available information, along with values of θ, in Table II.

Table II: Mooring Rope Dimensions

Ref. No.	Mooring Line Position	No. of Ropes or Wires	Dimensions (ft.) L_o	H Low Tide	H High Tide	ΔS_o	S_o Eq. (2)	θ^o	Rope Type*
1	Stern	4	90	14.5	17	1.0	92.10	30	8" Manila**
2	Stern	2	90	14.5	17	1.0	92.10	$22\frac{1}{2}$	2 coir 18" - 1 steel $3\frac{1}{2}$"
3	Stern	1	80	13.5	16	0.8	82.10	10	2 coir 18" - 1 steel $3\frac{1}{2}$"
4	Aft fore-spring	2	67.5	-4	-1.5	0.6	68.02	10	2 coir 18" - 1 steel $3\frac{1}{2}$"
5	Fore back-spring	2	60	4	-1.5	0.6	60.57	10	2 coir 18" - 1 steel $3\frac{1}{2}$"
6	Bow	1	65	8	10.5	0.8	66.34	10	2 coir 18" - 1 steel $3\frac{1}{2}$"
7	Bow	2	87.5	12	14.5	1.0	89.20	$22\frac{1}{2}$	2 coir 18" - 1 steel $3\frac{1}{2}$"
8	Bow	4	87.5	12	14.5	1.0	89.20	30	8" Manila**

* Coir-steel rope combinations represent single $3\frac{1}{2}$" steel wires attached to 18 ft. length loops of 18" coir rope.

** The four 8" manila ropes are assumed elastically equivalent to item 3 (or 6).

Elastic Properties of the Mooring Ropes

The length S assumed by a rope under the tension necessary to restrain the longitudinal movement of the ship to an amount X (Figs. 3 (a) and (b)) will exceed S_o by the amount of stretch dS. Since a rope under strong tension will be pulled effectively into straight line configuration,(2) it follows readily that the increase in horizontal projection, dL, is determinable from

$$dL = \sqrt{S^2 - H^2} - L_o . \tag{3}$$

In terms of dS, then

$$dL = \sqrt{(S_o + dS)^2 - H^2} - L_o . \tag{4}$$

From Fig. 3 (a) it will be seen that the longitudinal component, T_x of the horizontal component T_h of the tension in the rope at the ship is given by

$$T_x = T_h \left[1 - (Y_o/L)^2\right]^{\frac{1}{2}} \tag{5}$$

where Y_o is the distance of the center-line of the ship from the quay. Further since the longitudinal ship movement X is

$$X = L\left[1 - (Y_o/L)^2\right]^{\frac{1}{2}} - L_o\left[1 - (Y_o/L_o)^2\right]^{\frac{1}{2}} \tag{6}$$

we may perform the binomial expansions of the roots to two terms with sufficient accuracy, having regard to the values of the fractions Y_o/L and Y_o/L_o ($< 2/3$) and thus obtain

$$X = (L - L_o)\left[1 + \frac{1}{2}\,\frac{Y_o^2}{LL_o}\right] . \tag{7}$$

Dividing Eq. (7) by Eq. (5)

$$\frac{X}{T_x} = \frac{dL}{T_h}\left[\frac{1 + \frac{1}{2}(Y_o^2/LL_o)}{1 - \frac{1}{2}(Y_o/L)^2}\right] . \tag{8}$$

This result can be further simplified without serious error to

$$\frac{X}{T_x} = \frac{dL}{T_h}\left[1 + (Y_o/L_o)^2\right] \tag{9}$$

which may then be written

$$\frac{X}{T_x} = \frac{dL\,(1 + \sin^2\Theta)}{T_h} . \tag{10}$$

It can be shown that the difference between T_h and T, the full rope tension, is negligibly small even at comparatively low tensions.[2] Equation (10) therefore shows that the desired functional relationship between rope restraint and ship movement in the longitudinal direction can be obtained directly from the relationship between $dL\,(1 + \sin^2\theta)$ and T.

This relationship, it will be seen from Eq. (4), really resolves itself into knowing the dependence of rope extension dS on tension T, which must be found experimentally under conditions of repeated losding. Typical test results for 3-1/2-in. steel wire and 18-in. coir rope, obtained by plotting the center-lines of the repeated-loading hysteresis-loops from load-extension diagrams, are shown in Fig. 4 (a).

By suitably choosing values of T in increasing increments and allowing for the composite lengths of coir and wire rope in the coir-steel combinations (see footnote, Table II), the corresponding values of $dL\,(1 + \sin^2\theta)$ may be computed for each mooring line from Eq. (4), Table II and Fig. 3 (a), with results shown in Fig. 4 (b). These apply strictly to the several ropes leading aft; details for the forward leading ropes have not been evaluated as computational probes suggested that no important differences would be found.

From Fig. 4 (b) the cumulative sum of the tensions in all the ropes, $\sum T_x$, for any given value of X, can readily be obtained. Plotted against X in Fig. 5 the results yield the desired relationships between total restoring force R_x and ship movement X in the longitudinal direction. Since test results suggest that worn 18" coir rope is liable to failure at or near 15 tons, the capacity of the coir loops or 'springs' is not much above 30 tons. Accordingly Fig. 4 (b) shows that the backsprings (4) Table II - are liable to failure when the amplitude of ship movement reaches about 2 ft. The limits of failure are indicated in Fig. 5.

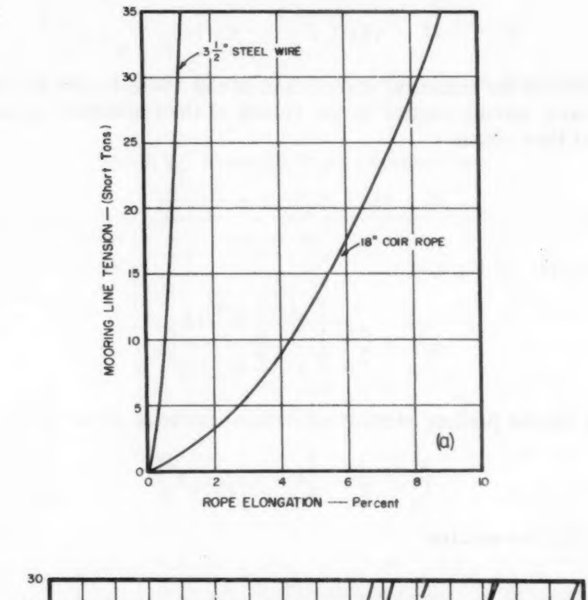


Fig. 4: Elastic properties of mooring ropes: (a) mean tension-elongation relationships under repeated loading; (b) tension-surge movement relationships for individual mooring lines

For convenience of formulation of the equation of surge motion of the ship curves of Fig. 5 have been fitted by an equation of the type

$$R_x = CX^n \tag{11}$$

where C is a constant and n a numerical exponent. The appropriate values of C and n found to give the best fit of Eq. (11) to the curves, for relatively large values of R_x, are given in Table III (see Fig. 5).

Table III: Spring Characteristics of Mooring Lines

Time May 5, 1952	Tidal Condition	Water Depth d (feet)	Spring Characteristics C (Tons - ft.$^{-n}$)	Spring Characteristics n
0700	Low	44	10.0	3.3
1200	High	46	26.1	2.4

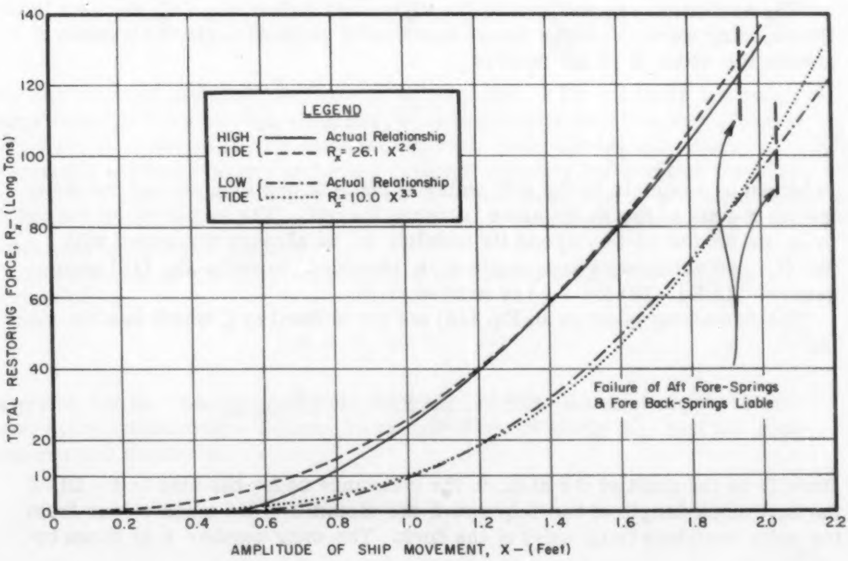


Fig. 5: Total restoring force relationship to surge-translation of the ship

Surge Motion of the Ship under Mooring Line Restraints

The equation of motion of the ship in the transverse standing wave system indicated in Fig. 2 has been developed elsewhere(9) and will merely be stated here in the form

$$\ddot{X} + 2\beta\omega\dot{X} + \omega^2 X^n = \zeta(\sigma \sin\phi - 2\beta\omega\cos\phi) \tag{12}$$

In Eq. (12) the symbols β and ω are defined by

$$\left.\begin{array}{ll} \text{(i)} & 2\beta\omega = \dfrac{N_x}{M'_x} \\[2ex] \text{(ii)} & \omega^2 = \dfrac{C}{M'_x} \end{array}\right\} \tag{13}$$

where N_x is a coefficient of linear damping and M'_x is the virtual mass of the ship for surge motion in the longitudinal direction. In turn M_x is defined as

$$M'_x = M + M''_x \tag{14}$$

in which M is the mass of the ship and M''_x the added mass resulting from the hydrodynamic pressures of the water on the ship in the direction of motion.

The excitation represented by the right hand side of Eq. (12) derives from the standing wave or seiche whose equation of vertical surface elevation η across the width B of the dock is

$$\eta = A\cos\frac{m\pi x}{B}\cos(\sigma t + \epsilon) \tag{15}$$

referred to an origin in the still water surface at one long side of the dock and an x-axis in the water plane traverse thereto. The amplitude of the seiche is A, its period $\tau(=2\pi/\sigma)$ and its nodality m, as already discussed with Eq. (1). An arbitrary phase angle ϵ is introduced to make Eq. (15) quite general. In Eq. (12) $(\sigma t + \epsilon)$ is written as ϕ.

The remaining quantity in Eq. (12) not yet defined is ζ which has the meaning

$$\zeta = \frac{Ag}{\sigma D}\,\frac{\sinh kd - \sinh ks}{\cosh kd}\,\frac{\sin k\ell}{k\ell}\sin kb \tag{16}$$

Here D is the draft of the ship, s the clearance under the ship (= d - D), ℓ the half block-length of the ship and b the distance of its mass center from the solid boundary (long side) of the dock. The wave number k is given by

$$k = \frac{m\pi}{B}\,. \tag{17}$$

Fortunately, the fact that the standing wave system is of long period permits of simplification of Eq. (16). Thus provided $\frac{md}{2B} < \frac{1}{20}$ we may introduce the approximations $\sinh kd \simeq kd$, $\sinh ks \simeq ks$, $\cosh kd \simeq 1$. For B = 2100 ft. and $d \simeq 45$ ft., this is valid to at least the 5th mode, whence, noting from Eqs. (1) and (17) that $\sigma \simeq k\sqrt{gd}$,

$$\zeta = A\sqrt{\frac{g}{d}} \cdot \frac{\sin k\ell}{k\ell} \sin kb \quad . \tag{18}$$

From Eq. (18) it is seen that the excitation must vanish when either $\sin k\ell$ or $\sin kb$ is zero. Thus the ship will tend to have no motion if

$$\left.\begin{aligned} &\text{(i)} \qquad \ell = B/m \\ &\text{(ii)} \qquad b = B/m \end{aligned}\right\} \tag{19}$$

This implies that the ship will lie quietly if either its half-length ℓ of its distance b from the end boundary is an integral submultiple of the width of the dock B. On applying the test we find $B/\ell = \frac{2100}{240} = 8.76$ and $B/b = \frac{2100}{590} = 3.56$ and conclude that the ship was inevitably prone to surging.

The circumstances in which the ship would be liable to maximum excitation would be such as to make either $\sin k\ell$ or $\sin kb$ unity, for which

$$\left.\begin{aligned} &\text{(i)} \qquad \ell = \frac{pB}{2m} \\ &\text{(ii)} \qquad b = \frac{pB}{2m} \end{aligned}\right\} \quad p = (1, 3, 5 \ldots) \tag{20}$$

For the various modes of oscillation shown in Fig. 2 (m = 1 to 7), the relative magnitudes of the amplitude factor ζ/A of the forcing function are shown in Table IV. It is seen that the uninodal, binodal, quinquinodal and possibly sextinodal seiches emerge as having greatest capacity for exciting the surge motion of the ship, a fact which may be inferred from Fig. 2, since for these modes of oscillation the ship center lies closest to the nodes where horizontal velocities of flow are highest.

The solution to Eq. (12), representing the forced oscillation, may be stated[3,9] as

$$X = X_1 \cos(\phi - \alpha) \tag{21}$$

wherein, for the case of negligible damping ($2\beta \to 0$) - as would apply to the case being considered - the maximum amplitude of surge X_1 and the phase angle α are defined by

$$\left.\begin{aligned} &\text{(i)} \qquad X_1^n \Delta(n) - (\sigma/\omega)^2 X_1 = -\frac{\zeta}{\omega}(\sigma/\omega) \quad , \\ &\text{(ii)} \qquad \tan\alpha = \frac{\sigma}{2\beta\omega}\left[\frac{(2\beta)^2}{X_1^{n-1}\Delta(n) - (\sigma/\omega)^2} - 1\right] . \end{aligned}\right\} \tag{22}$$

Table IV: Comparative Magnitudes of Modes of Excitation

Mode or Nodality of Seiche	Wave Number	Ship-Position Relationships			$(\zeta/A)\,(\mathrm{sec}^{-1})$	
m	k	$\frac{\sin k\ell}{k\ell}$	sin kb	$\frac{\sin k\ell}{k\ell}\sin kb$	Low Tide	High Tide
1	1.50	0.98	0.77	0.76	0.65	0.63
2	2.99	0.95	0.98	0.93	0.79	0.78
3	4.49	0.90	0.47	0.43	0.36	0.36
4	5.99	0.83	0.38	0.32	0.27	0.26
5	7.48	0.74	0.96	0.71	0.60	0.59
6	8.98	0.64	0.83	0.53	0.45	0.44
7	10.48	0.53	0.10	0.05	0.05	0.04

In Eqs. (22) $\Delta(n)$ is a numerical function of n only, whose values have been given elsewhere.(3,9) For present purposes it is sufficient to state that the values of $\Delta(n)$ applicable to the n-values found in Table III are as given later in Table V.

From Eqs. (13, ii) and (18) the value of $\frac{\zeta}{\omega}$ may be written

$$\frac{\zeta}{\omega} = A' \sqrt{\frac{C_M W}{C d}} \tag{23}$$

where

$$\left.\begin{aligned} &\text{(i)} && A' = A\,\frac{\sin k\ell}{k\ell}\sin kb \\ &\text{(ii)} && C_M = \frac{M'_x}{M} = \left[1 + \left(\frac{M''_x}{M}\right)\right] \\ &\text{(iii)} && M''_x = C_x\,\frac{\pi}{8}\,\rho B^2 D\ . \end{aligned}\right\} \tag{24}$$

Here A' is the effective seiche amplitude, W the dead weight of the ship, C_x an inertia coefficient whose value depends on the ship beam-length ratio $(a/2\ell)$, and ρ the mass density of water. The block-length of the ship required to displace 18,000 tons of water is $2\ell = 350$ ft. and the beam-length

ratio is thus $a/2\ell = 0.19$, for which C_x in Eq. (24, iii) is about 2.0.(9) For the dimensions of the ship, then, M_x'' is found to be 1.99×10^5 and $M = 12.52 \times 10^5$ slugs. Thus $C_M = 1.16$ and the forcing function term $(\frac{\zeta}{\omega})$ of Eq. (23) assumes the values listed in Table V for the values of C and d given in Table III.

Table V: Parameters in the Equation of Surge Motion

Time (May 5, 1952)	Tidal Condition	Water Depth d (ft)	Rope Constants			Equivt. Natural Frequency ω Eq. (13 ii)**	$\frac{\zeta}{\omega A'}$ Eq. (23)***
			n	C*	Δ(n)		
0700	Low	44	3.3	10.0	0.725	0.127	6.75
1200	High	46	2.4	26.1	0.805	0.200	4.17

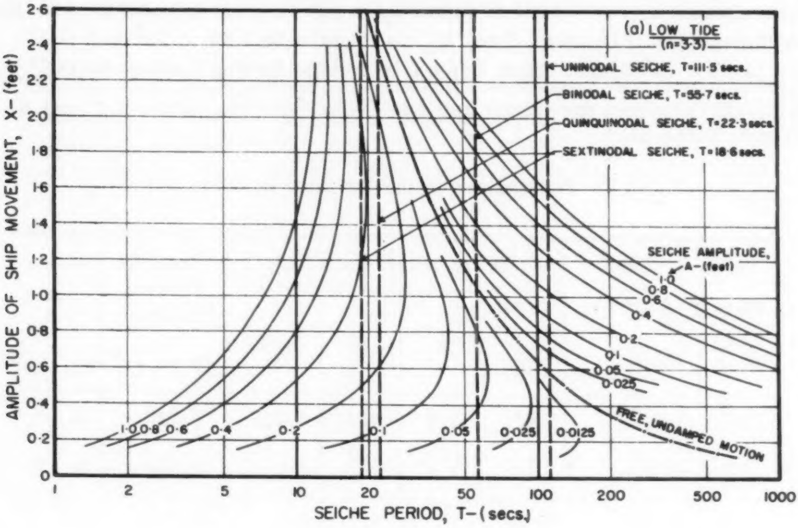
* tons ft^{-n} ** $ft^{\frac{1-n}{2}} sec^{-1}$ *** $ft^{\frac{n-1}{2}}$

In solving Eq. (22 i) it is convenient to write $\xi = \sigma/\omega$ and derive the roots of the quadratic in ξ in terms of A' and X_1. Thence by choosing different values of A' and X_1 the families of curves shown in Fig. 6 are obtained. The inter-related quantities ξ, A' and X_1 are there plotted as X_1 versus τ with isolines of A', τ being derived directly from $\sigma = (2\pi/\tau) = \omega\xi$.

The undamped free oscillation, for which the excitation ζ in Eq. (22 i) is zero, is amenable to exact solution(3,9) and yields the 'backbone' curves of Figs. 6 (a) and (b).

Response of the Ship to the Various Modes of Excitation

It was shown in Table IV that the seiches most likely to influence the ship and promote surging in virtue of the ship's length and position in the dock would be the fundamental transverse oscillation in the dock and its second and fifth harmonics (Fig. 2). Vertical ordinates drawn in Figs. 6 at the values of τ applicable to these modes (Table I) define the behavior-patterns of the ship at these individual frequencies of excitation. As an example, consider a binodal seiche $\tau = 55.7$ secs prevailing at low tide, Fig. 6 (a). Assuming that the effective amplitude A' were to increase from zero at a reasonably slow rate in time, the ordinate tells us that at the amplitude of $A' \simeq 0.065$ ft. the amplitude of ship movement would be $X_1 \simeq 0.58$ ft. Then suddenly, if A' increased further, the ship movement would jump through an instability to 1.2 ft. (on the upper side of the 'backbone' curve). This trend is illustrated in another way in Fig. 7 (d), wherein the further increase of seiche amplitude A' at low tide is shown to follow the uppermost curve. The corresponding curve obtained from the ordinate at $\tau = 54.5$ secs at high tide (Fig. 6 (b)) forms the lowest curve of Fig. 7 (d). Interpolated between are curves such as might apply at hourly intervals between low and high tides.



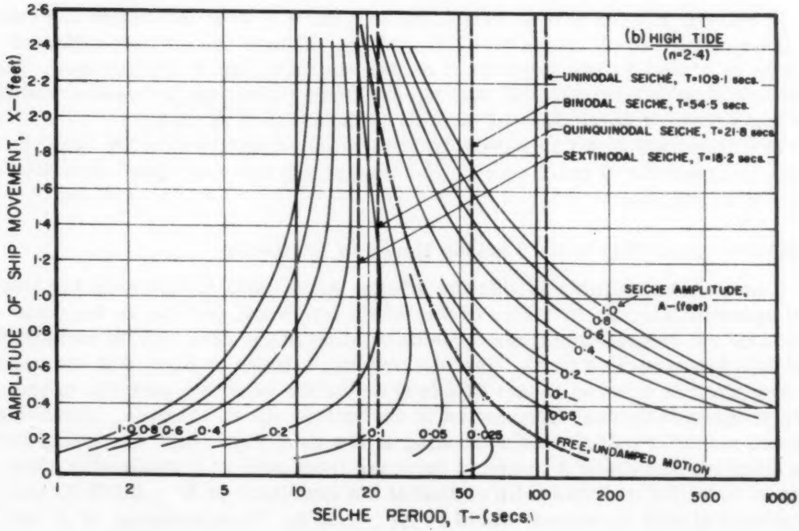


Fig. 6: Characteristics of non-linear surge motion of ship: (a) at low tide; (b) at high tide

Figs. 7 (c) and 7 (e) correspond to the conditions of a uninodal seiche and quinquinodal seiche respectively and are derived similarly from Figs. 6. The curves of Fig. 7 (e) are rather notably different from (c) and (d) because of the larger jump tendencies to which the ship is subject as a consequence of the non-linear instability.

From Fig. 7 (b), the amplitudes A of the uninodal, binodal and quinquinodal seiches, prevailing on the hour, have been recorded and tabulated in Table VI. The effective seiche amplitudes, A', derived directly from Eq. (24 i) are also given from information provided in Table IV. The values of A' given in columns (6), (7) and (8) have been plotted in Figs. 7 (c) (d) and (e) respectively to yield the circled points on the hourly lines of X_1 versus A'. The connecting heavy lines define the continuous response of the ship in surge to the increasing amplitudes A' which occurred with the rise of tide.

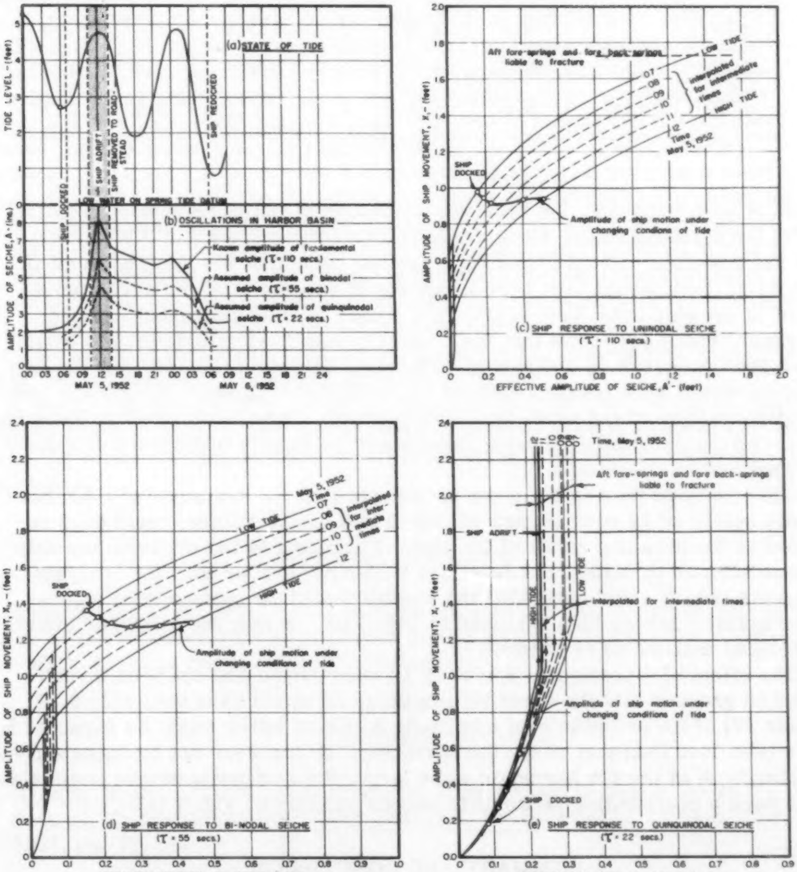


Fig. 7: Continuous surge response of oil tanker to transverse seiches

Table VI: Probable Real and Effective Amplitudes of Seiches

Time (May 5, 1952)	Tidal Condition	Probable Real Ampli, A(ins)			Effective Ampli, A' (ins)		
		m = 1	m = 2	m = 5	m = 1	m = 2	m = 5
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0700	low	2.6	2.0	1.4	2.0	1.9	1.0
0800		3.0	2.4	1.8	2.3	2.2	1.3
0900		3.6	2.9	2.2	2.7	2.7	1.5
1000		4.6	3.6	2.8	3.5	3.3	2.0
1100		6.5	4.7	3.7	4.9	4.4	2.6
1200	high	8.0	6.0	4.4	6.1	5.6	3.1

It is seen from Fig. 7 (c) that the amplitude of surge movement resulting from the uninodal seiche alone would not have exceeded about 0.95 ft. throughout the tide rise. From Fig. 7 (d) it is also clear that the second harmonic seiche of itself would have been incapable of causing a larger surge movement of the ship than $X_1 \simeq 1.3$ ft. In the case of the quinquinodal seiche, however, Fig. 7 (e) shows that the ship response to amplitude growth would have been progressive and that at some time between 1100 and 1200 the ship surging would have reached critical instability and caused over-loading and probable failure of the fore- and back-springs. Once these mooring lines had ruptured the remaining mooring lines would be rapidly overloaded and could be expected to follow suit.

This analysis then would seem to indicate that the existence of a 5th harmonic seiche of 22 secs period, of only 3 to 4 ins amplitude, could have resulted in the breaking adrift of the ship. According to the evidence the ship was in serious difficulties at least one half-hour earlier than the analysis suggests, which could mean that the amplitudes of the quinquinodal seiche were actually larger than assumed in Fig. 7 (b), or that the combined effects of several seiches were at work.

The effect of a sextinodal seiche of 18 secs period has not been investigated on grounds that its effective amplitude A' would have been only 0.53 (Table IV) of its probable real amplitude A, which latter might be expected to have been less than that of the 5th harmonic. If this were not the case and the amplitude A of the 6th harmonic were large, the sextinodal seiche could also have been a contributing factor with results similar to Fig. 7 (e).

SUMMARY AND CONCLUSION

The results of this study show that the 18,000-ton oil tanker, in virtue of her berthing location, her length and the characteristics of her mooring ropes, was innately responsive to longitudinal surge motion excited by the

fundamental transverse seiche in the dock and its second, fifth and possibly sixth harmonics. Indications are that the first two of these oscillations, despite their comparatively large amplitude growth on the rising tide, would not have stimulated any very serious longitudinal motion of the ship; in fact the motion remained indifferent to the increasing magnitude of these sea disturbances. On the other hand the 5th harmonic transverse seiche, even with quite modest amplitude growth (up to 4 ins.), would have excited an increasing surge movement of the ship up the point that the ship would suddenly and violently have lunged fore and aft and thereby strained the 18" coir springs of the mooring lines beyond their limits of recovery. It must be supposed that the co-existence of these several modes of sea oscillation could have greatly aggravated and hastened the failure of the mooring lines.

The reasons for the ship's critical response to these lesser and higher-frequency disturbances must be sought in the peculiarities of the characteristic curves (Figs. 6), relating seiche amplitude A' and period τ with surge amplitude X_1, which form the solution of the non-linear equation of longitudinal ship motion. Obviously of great importance is the resiliency of the mooring lines as reflected in the values of C and n. These values largely dictate how the 'backbone' curve will be disposed in relation to the exciting periods and how steeply it will rise with increase of X_1. Clearly, if the 'backbone' curves of Figs. 6 were to rise less sharply the conditions could be considerably improved to the extent that at the period of the 5th harmonic (22 secs) the instability jump might conceivably be contained within values of X_1 which would not overstrain the ropes. The possibility that still higher frequency excitations would then become critical could be ruled out on the basis that there are inherent tendencies towards evanescence of increasingly higher harmonics, quite apart from the decay features of the factor ($\sin k\ell / k\ell$) with increasing nodality m, (exhibited in column 3 of Table IV), which tend to reduce the effective seiche amplitude A'.

It would thus seem that in harbors subject to surging much might be done towards alleviating dangerous conditions for ships of certain sizes at certain berths by judicious design of the mooring systems to be used in given circumstances.

ACKNOWLEDGMENT

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List of Symbols

a	beam or breadth of ship
A	amplitude of vertical surface elevation of seiche
A'	effective amplitude of seiche, Eq. (24 i)
b	distance of center of ship from one end of dock
B	width of dock or basin
C	non-linear spring constant
C_M	coefficient of virtual mass, Eq. (24 ii)
C_x	inertia coefficient in surge, Eq. (24 iii)
d	water depth
D	draft or wetted depth of ship
g	acceleration due to gravity
H	height of rope fairleads on ship above quay level
k	wave number ($= \frac{m\pi}{B}$)
ℓ	half-length of ship represented as a rectangular block ($= M/2\rho BD$)
L	horizontal projection of stretched mooring line
L_o	horizontal projection of mooring line for stationary ship in equilibrium position
dL	$(L - L_o)$

n	numerical exponent depending in value on rope characteristics
m	integer (1, 2, 3 . . .) defining nodality or harmonic mode of seiche
M	mass of ship
M'_x	virtual mass of ship for surge motion in x-direction ($= M + M''_x$)
M''_x	added hydrodynamic mass for surge motion in x-direction
N_x	linear damping coefficient in surge motion
p	odd integer (1, 3, 5 . . .)
R_x	total restoring force in x-direction from all mooring lines acting bowwards (or sternwards)
s	clearance between sea bed and ship keel ($= d - D$)
S	length of stretched mooring line
S_o	length of mooring line for stationary ship in equilibrium position
ΔS_o	excess of S_o over $\sqrt{L^2 + H^2}$
t	variable time
T	tension in mooring line at the ship
T_h	horizontal component of T
T_x	component of T_h in x-direction
W	dead weight of ship
x	variable horizontal distance from origin in still water surface at one long side of dock, transverse thereto
X	variable horizontal longitudinal (surge) translation of mass-center of ship from rest position
X_1	amplitude of forced oscillation of ship in surge
$\dot{X}$, $\ddot{X}$	variable horizontal surge velocity and acceleration of mass-center of ship
Y_o	horizontal distance of center-line of ship from quay wall
α	phase angle, Eqs. (21) and (22)
β	damping factor, Eq. (13 i)
$\Delta(n)$	function of n only
ϵ	phase angle
ζ	excitation factor, Eq. (12) and (16)
η	variable vertical elevation of water surface, with reference to still water level
θ	angle between horizontal projection of mooring line and side of quay (Fig. 3)

ξ	ratio σ/ω
π	universal constant (3.14159 . . .)
ρ	mass density of water
σ	angular frequency of seiche (= $2\pi/\tau$)
$\sum$	summation symbol
τ	period of seiche, Eq. (1)
ϕ	angle (= $\sigma t + \epsilon$)
ω	equivalent natural frequency of non-linear surge oscillation of ship, Eq. (13 ii)

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

THE EFFECT OF SEICHES AT CONNEAUT HARBOR[a]

Discussion by Basil W. Wilson

BASIL W. WILSON,[1,2] F. ASCE.—In the absence of specific examples of the types of accidents occurring to shipping in Conneaut Harbor, the writer finds himself unable to infer from the author's paper just what kind of connection with the lake seiches these accidents are supposed to have. Thus, conceding the point that a correlation exists between the water-level rises at Buffalo and the prevalence of accidents at Conneaut, this does not necessarily indicate that the fundamental lake seiche is in any way responsible for particular accidents. Since strong winds are required to initiate water superelevation downwind, along with attendant inertial oscillations or seiches over the entire lake, the same winds are capable of generating waves and local short period inertial oscillations within the harbor which alone could be responsible for the accidents, depending of course on their nature. It is noticed that on page 39 the author states that "frequent accident reports cite surging action as the cause of damage" in the inner harbor and the slip. In these cases it can then be definitely said that the exciting disturbances would be of short period, say less than 2 minutes, with no possible relation to the long 15-hour lake seiche. The literature on ship surging is now quite considerable and this point is fairly well established (Wilson, 1950, 1957, 1958 (i) and (ii), 1959; O'Brien, 1954; Russell, 1957, 1959; Joosting, 1957; O'Brien and Kuchenreuther, 1957 (i) and (ii), 1958, 1959; Wiegel et al., 1957). It is unfortunate that the author does not present any examples of actual water level records in Conneaut Harbor itself, which might indicate the existence of oscillations within the breakwaters or the inner harbor.

From the author's Fig. 1 it would appear that the length of the inner harbor from its mouth to the head of the slip is of the order of 4175 ft. and that the average depth of water is about 23 ft. Since this is effectively a long rectangular canal opening at one end on a larger body of water, the modes of longitudinal oscillation to which it would be susceptible will be given fairly accurately (Lamb, 1932 Edn., art. 179) by

$$T_n = \frac{4\ell}{n\sqrt{gd}} \tag{14}$$

where n is the harmonic order of the mode (n = 1, 3, 5 . .), T_n the period of the mode, ℓ the length of the canal, d its depth and g the acceleration due to

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a. Proc. Paper 2067, June, 1959, by I. A. Hunt.

1. Contribution from the Dept. of Oceanography and Meteorology, Agri. and Mechanical College of Texas, Oceanography and Meteorology Series.
2. Prof. (Eng. Oceanography), A. & M. College of Texas, College Station, Tex.

gravity. For the dimensions quoted, the natural periods T_n would be (Table 3):

Table 3: Natural Periods of Longitudinal Oscillation: Inner Harbor

Harmonic Order n	Natural Period T_n (mins)
1	10.23
3	3.41
5	2.04
7	1.46
9	1.14

Of these modes only the 5th, 7th and 9th harmonics are within the period range likely to be critical for ships.

In the transverse direction across the canal the fundamental natural period of oscillation would vary with the width according to the author's Eq. (1). Likely values are given in Table 4.

Table 4: Natural Periods of Transverse Oscillation: Inner Harbor

Width of channel (feet)	Fundamental Natural Period, T_1 (secs)
200	14.7
250	18.4
300	22.0
700	51.5

These periods are all within the range capable of exciting ship disturbances.

Regarding the possibility that cross-currents arising from the fundamental lake seiche could be of consequence to the navigation of ships through the harbor entrances, the writer believes that the author has considerably over-estimated the magnitudes likely to prevail at Conneaut. The source of the author's Eq. (5) is the equation of motion for a water particle (Sverdrup et al., 1942).

$$\frac{\partial^2 \xi}{\partial t^2} = - g\frac{\partial \eta}{\partial x} \tag{15}$$

for the assumed solution

$$\xi = \xi(x) \cos \sigma t \tag{16}$$

where $\zeta(x)$ is a function of distance x along the axis of the lake, σ the seiche frequency (= $2\pi/T$) and t variable time. Thus, substitution of Eq. (16) in (15) yields the author's Eq. (16). What seems to have been overlooked by the author is that his ζ is a function of x and that its applicable value at Conneaut may be less than the value at the node of the seiche; also that its value depends on the component of water level change, (not the whole water level change) attributable to the fundamental lake seiche.

The continuity equation which must be satisfied is

$$\eta = -\frac{1}{b}\frac{\partial(S\xi)}{\partial x} \tag{17}$$

where S is the vertical cross sectional area of the lake transverse to the longitudinal axis and b the surface breadth, (both functions of x). For an assumed solution of the form

$$\eta = \eta(x) \cos \sigma t \tag{18}$$

Eq. (17) may be integrated and reduced to the form

$$\xi_X = \frac{1}{S_X}\int_0^X b(x)\,\eta(x)\,dx \tag{19}$$

The function $\eta(x)$ is here the longitudinal surface profile of the lake at the moment of maximum seiche amplitude ($\cos \sigma t = 1$). The quantities ζ_X and S_X refer to the values of $\zeta(x)$ and $S(x)$ applying at any point (say, Conneaut) whose distance from the reference origin, taken at Toledo, is x = X. Eq. (19) may conveniently be restated as

$$\xi_X = \frac{(L/D_o)}{(b_X/b_o)(D_X/D_o)}\int_0^{X/L} \frac{b(x/L)}{b_o}\cdot \eta\left(\frac{x}{L}\right) d\left(\frac{x}{L}\right) \tag{20}$$

wherein b_o and D_o are respectively the mean breadth and mean depth for the lake, b_X and D_X the breadth and mean depth at x = X, and L the length of the lake.

To evaluate ζ_X in a typical instance we need to be careful as to how we define the lake seiche. Thus from the author's Fig. 2 it is possible to show that the water level variations at Buffalo and Toledo were each made up of four principal components A, B, C, D and A', B', C', D', as shown in Fig. 6. Of these A and A' are respectively the mean set-up and draw-down of lake level resulting from wind stress under quasi-steady-state conditions. The dynamic oscillations superimposed on the curves A are the remaining curves B, C, D and B', C', D' of which C, D and C', D' are probably local seiches peculiar to the Buffalo and Toledo ends of the lake. Thus at Buffalo there would appear to be a local oscillation (C) of about 6 hours period with a second harmonic (D) of about 2 to 3 hours period, superimposed on the fundamental lake seiche (B) of 15 hours period. It seems likely that the oscillations C and D are connected with the east end of Lake Erie between Buffalo and Long Point Island. Similarly at Toledo there would seem to be evidence of an 8 to 9 hours oscillation (C') and one of about 5 to 6 hours (D') which could be related to the oscillating characteristics of the lake between Toledo and Pelee Island. In the

absence of more information it cannot, of course, be said definitely that the oscillations C, D and C', D' are not just second and third harmonics of the fundamental lake seiche. If they are, their prominence is probably due to a measure of agreement between the natural periods of the east and west end bights and the second and third harmonic periods for the whole lake.

Essentially then, the fundamental lake seiche of 15 hours period prevailing on 3-4 November, 1955, is represented by the curves B and B' of Fig. 6. The longitudinal variation of water level along the lake for this component alone at 1030 on November 3, 1955, would then have conformed sensibly with the profile shown in Fig. 7(a) which is inferred as to general shape, from the author's Fig. 3 and from the condition that

$$\int_0^L b(x)\ \eta(x)\, dx = 0 \tag{21}$$

as prescribed by continuity.

By recourse to the breadth and depth distributions along the lake given in Figs. 7(b) and (c) respectively, conveniently adopted from Keulegan (1953), Eq. (20) has been integrated graphically to give the values of the horizontal particle displacements ζ_X at any point along the lake. At Conneaut (x/L = 0.57) the surge movement is thus found to be of the order $\zeta_X = 1.16$ miles.

To determine the maximum surge velocities, u_o, we have merely to differentiate Eq. (16) with respect to time and set $\sin \sigma t = 1$. This yields

$$u_o = \left(\frac{\partial \xi}{\partial t}\right)_{max} = \sigma \xi(x) = \frac{2\pi}{T}\, \xi(x) \tag{22}$$

The values of $(u_o)_X$ thus obtained for T = 15 hours are shown in Fig. 7(d) from which it is seen that the maximum surge current at Conneaut is only about 0.6 ft/sec. This is very much less than the value of 1 ft/sec. found by the author and virtually rules out the possibility that seiche currents are of a dangerous order of magnitude. Reference to Fig. 6, moreover, shows that the maximum seiche current would have occurred at about 0700 and 1300, November 3rd, 1955, at times when there would have been very little longitudinal transfer movement of water from set-up and draw-down. Any superimposed currents from second and third harmonics near the middle of the lake would probably have been feeble and likely to be of little consequence in any case because of tendencies to annul each other.

The writer thus finds himself unable to agree with the author, on the available evidence, that currents attributable to the fundamental lake seiche are a very serious factor in the troubles experienced at Conneaut. In the absence of more definite facts the writer would hesitate to question the author's contention that reorientation of the east pier would necessarily improve matters, but again this view would now seem to be rather poorly supported by the author's main thesis. It would appear to the writer that more specific enquiry into the precise nature of the accidents and the water oscillations causing them would be a very desirable feature of extended studies.

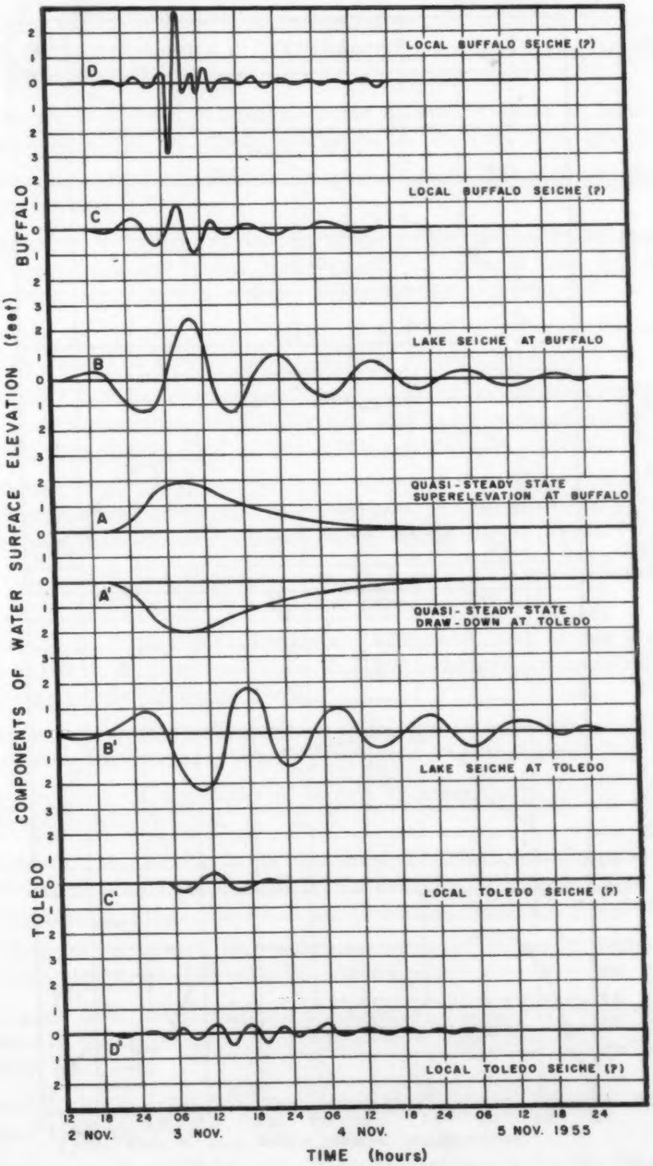

Fig. 6. Apparent Components of Simultaneous Water Surface Elevation at Buffalo and Toledo

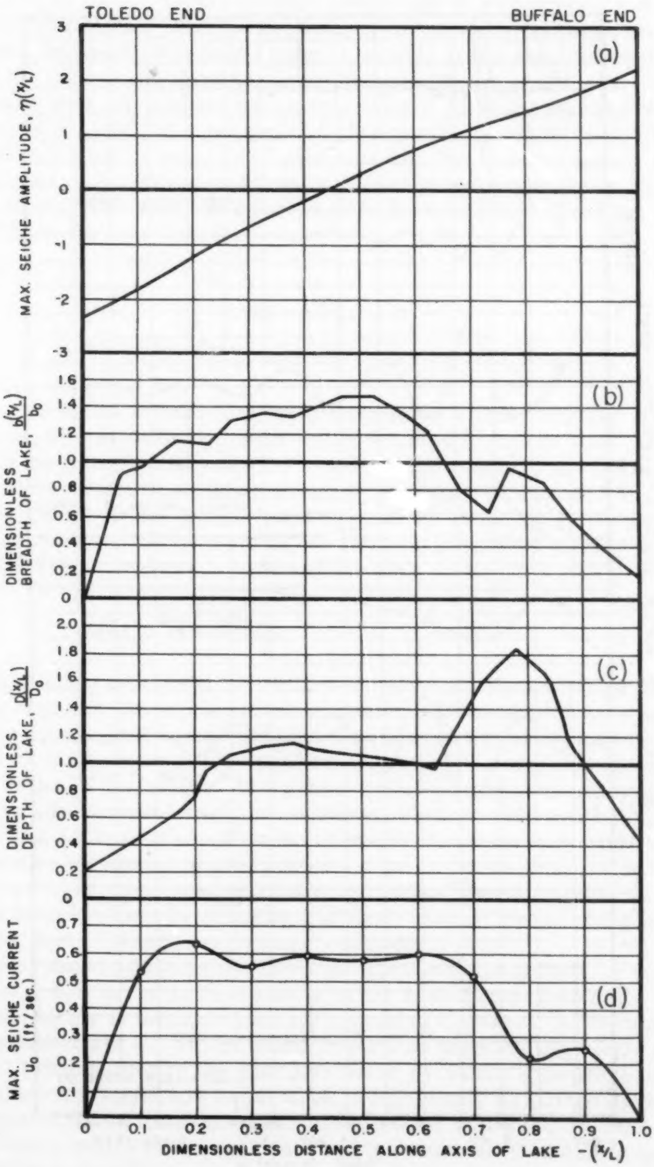

Fig. 7. Functions dependent on distance along the lake-axis.

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17. Wilson, B. W. (1959); Critical Non-Linear Motion of a Moored Ship under Surge Action; Paper due for presentation Midwestern Conf. Fluid and Solid Mechs., Univ. Texas, Austin, Sept. 1959.

SUBSIDENCE PROBLEM IN THE LONG BEACH HARBOR DISTRICT[a]

Discussion by A. F. Benscheidt

A. F. BENSCHEIDT,[1] F. ASCE.—Mr. Berbower, because of publishing limitations, obviously needed to choose from among the many interesting and significant facts about the Long Beach harbor subsidence. Thereby he may have left the casual reader with the impression that the designer must cope with random and somewhat unpredictable horizontal movements.

While minor tension-compression reversals do occur, the pattern of the surface strains does follow U. S. Grant's flat plate analogy, first developed in a 1948 study for the Harbor Department.(1)

Grant viewed the subsidence area as similar to an elliptical thick flat plate, about 2 miles in diameter and 3000 feet thick, clamped around the edges, and deformed by its own deadweight as the supporting pressures are reduced. Later researches raised some questions about this approach, but the concept serves the designer well as a guide to what horizontal strains are to be expected in the subsidence area.

Thus in the central mile of the actual subsidence bowl, as in the flat plate, the top surface is in compression. Conduits push into manholes, piping buckles, and buildings are shortened, regardless of their orientation.

At about 3500 feet horizontally from the major axis of the bowl, corresponding approximately to the 12' subsidence isobase, Fig. 1, a line of counterflexure is found.(2) Though the angular declination is there a maximum, it is an area of decreased strain. However, since the ground surface is moving toward the center of the bowl, concentric circles must become smaller so that structures lying more or less parallel to the isobase line are subjected to compression. From Fig. 3C it is evident that an ellipse at the 12' isobase (roughly the 10' isobase on Fig. 3C) originally 30,000 feet in circumference might well be reduced 20 feet or 0.07 feet per 100. Aside from this dominant trend, local variations make it difficult to anticipate conditions in this zone.

Outside of the counterflexure zone, structural elements near the ground surface, and lying radial to the center of the bowl, are consistently in tension, as evidenced by tension breaks in utility lines and stretching of buildings.

Structures oriented roughly parallel to the isobase lines undergo compression, again because they lie on a line of decreasing length. Compression of buildings on Pier A, the compression of the Ford Motor Company assembly plant, and the binding of the lift span of the Commodore Heim bridge(3) as the towers move together are typical of this condition.

At distances of two miles or more from the vertex little trouble is experienced but horizontal movements toward the center are still significant,

a. Proc. Paper 2068, June, 1959, by R. F. Berbower.

1. Chf. Assoc. Engr., George F. Nicholson, Cons. Engr., Long Beach, Calif.

suggesting that vertical subsidence at the fringes of the bowl may in part be a secondary effect through the "stretching" of the upper half of the flat plate.

The similarity with the flat plate is not so evident under the surface of the earth, but a shear plane at about 1500 foot depth has destroyed many wells, particularly after any slight earthquake that hastens the return to equilibrium.

REFERENCES

1. Long Beach Harbor Department, "Analysis of Horizontal Movements in the Long Beach Harbor Area", July 1948, by U. S. Grant.
2. Surface Subsidence and Remedial Measures at the Long Beach Naval Shipyard, Long Beach, California, 1957, by Charles H. Neel, M.S.
3. Subsidence in Long Beach—Terminal Island—Wilmington, California—Pacific Fire Rating Bureau—March, 1958.

LABORATORY INVESTIGATION OF RUBBLE-MOUND BREAKWATERS[a]

Corrections

CORRECTIONS.—On page 94, in the line 8 in the first paragraph under the heading "Introduction," change the words "experimental coefficients, or the" to "experimental coefficients, of the".

On page 104, just prior to Eq. (16), the following material should be inserted:

Results of Stability Tests

No-Damage Conditions

Data obtained from stability tests of quarry-stone and tetrapod-shaped armor units for the no-damage criterion are shown in Fig. 6 in the form of a log-log plot, with the stability number as the ordinate, cot as the abscissa, and the shape of the armor unit as the parameter. These data consist of experimentally determined design-wave heights and corresponding calculated stability numbers, as functions of breakwater slope and shape of armor unit. Data concerning quarry-stone armor units were obtained for breakwater sections of the type shown in Fig. 3, and the design-wave heights were determined for the no-damage and no-overtopping criteria. Data concerning tetrapods, using the no-damage and no-overtopping criteria, were obtained for breakwater sections of the type shown in Fig. 4. Data were also obtained for a breakwater section of the type shown in Fig. 5, using the no-damage criterion. The crown of the latter breakwater section was designed for overtopping.

Analysis of the test data indicated that, for the conditions tested, the effects of the variables d/λ and H/λ on the stability of armor units are of second order in importance compared with the effects of breakwater slope and shape of armor unit. A formula for determining the weight of armor units necessary to insure stability of rubble-mound breakwaters of the types tested, and in relatively deep water, can be obtained from the equation of the approximate best-fit lines in Fig. 6. The lines AB and MN were drawn through the data points using a slope of one-third to simplify the derived formula. The equation of a straight line on log-log paper is of the form $y = ax^b$ where a is the y intercept at $x = 1$, and b is the slope of the line. The equation of lines AB and MN, therefore, is . . .

a. Proc. Paper 2171, September, 1959, by Robert Y. Hudson.

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

VOLUME 84 (1958)

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867(ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876(WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

VOLUME 85 (1959)

JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926(PL1), 1927(HW), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958(CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986(IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998(SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007(HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056(HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065(ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087(SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096(EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134(SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158(IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176(WW3), 2177(HW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184(WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205(PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214(SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223(ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240(PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248(SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278(HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287(PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296(SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322(ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330(CP2).

c. Discussion of several papers, grouped by divisions.

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DECEMBER 1959 — 46 NO. WW 4
VOLUME 85 PART 2

Your attention is invited

NEWS OF THE WATERWAYS AND HARBORS DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

JOURNAL OF THE WATERWAYS AND HARBORS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

December, 1959

NEW CHAIRMAN OF DIVISION EXECUTIVE COMMITTEE

Professor Joe W. Johnson from the University of California was elected Chairman of the Waterways and Harbors Executive Committee at the October meeting. Professor Johnson has been a member of the Executive Committee for two years. He succeeds as Chairman Col. Lawrence B. Feagin, Chief of the Operations Division of the Mississippi River Commission, who remains on the Executive Committee. Col. Feagin also remains as the Division's representative on the ASCE Coordinating Committee on Water Resources.

Mr. Richard O. Eaton, Chief Technical Advisor, Beach Erosion Board, Corps of Engineers, was elected Vice-Chairman of the Committee. Mr. Eaton has been a member of the Executive Committee for one year.

At the October meeting, Mr. Roger H. Gilman, Director of Port Development for the Port of New York Authority, retired from the Executive Committee. Mr. Gilman is succeeded by Mr. Evan W. Vaughan of Parsons, Brinkerhoff, Hall and Macdonald, who has been Secretary of the Executive Committee since October 1956 and prior to that date was Chairman of the Division Committee on Publications. Mr. Gilman remains as the Division's representative on the ASCE Coordinating Committee on Transportation.

COMMITTEE ON SESSION PROGRAMS SPONSORED TEN SESSIONS DURING YEAR

The Division Committee on Session Programs under the able chairmanship of Mr. Raymond W. Sauer handled arrangements for ten sessions of the Division at three conventions during the 1958-1959 year. The programs consisted of:

a. 1958 New York Convention - Robert J. Winters, Local Contact Member

Attendance - Max. 125, Avg. 95

Note: No. 1959-46 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, WW 4, December, 1959.

Sessions - 3

Papers were sponsored by:

Committee on Research - 2
Committee on Ports and Harbors - 2
Committee on Coastal Engineering - 4
Committee on Navigation and Flood Control Facilities - 4

Presided over by Roger H. Gilman

b. 1959 Loss Angeles Convention - Carroll T. Newton, Local Contact Member

Attendance - Max. 175, Avg. 96

Sessions - A total of 4 sessions and one harbor inspection trip were sponsored by the Waterways and Harbors Division. The Hydraulics Division, City Planning Division and Power Division joined us in sponsoring one each of 3 sessions.

Papers were sponsored by:

Committee on Navigation and Flood Control Facilities - 2
Committee on Ports and Harbors - 7
Committee on Coastal Engineering - 4

Presided over by Joe W. Johnson

c. 1959 Cleveland Convention - Richard M. Gensert, Local Contact Member

Attendance - Max. 50, Avg. 40

Sessions - Three technical sessions and one harbor inspection trip.

Papers were sponsored by:

Committee on Research - 2
Committee on Regulations and Stabilization of Rivers - 1
Committee on Navigation and Flood Control Facilities - 3
Committee on Coastal Engineering - 3

Presided over by R. O. Eaton

As of October 1959, Mr. Mark S. Gurnee, Chief, Operations Division, Office of the Chief of Engineers, Corps of Engineers, succeeds Ray Sauer as Chairman of the Division Committee on Session Programs. The job Ray has done in developing excellent programs of interest and value to the profession has been outstanding.

FOR YOUR CALENDAR

January 19-20, 1960	Princeton Conference
March 7-11, 1960	ASCE, New Orleans Convention
June 19-23, 1960	ASCE, Reno Convention
October 9-13, 1960	ASCE, Boston Convention

April 10-15, 1961	ASCE, Phoenix Convention
October 16-20, 1961	ASCE, New York Convention
February, 1962	ASCE, Houston Convention
May, 1962	ASCE, Omaha Convention
October 15-19, 1962	ASCE, Detroit Convention

WATERWAYS AND HARBORS DIVISION PUBLISHED FOUR JOURNALS

The Division Committee on Publications, under the able direction of Jay V. Hall, Jr., Chief, Engineering Division, Beach Erosion Board, handled 41 papers during the 1958-1959 year, and published four Waterways and Harbors Division Journals:

1. Manuscripts carried over from year 1957-58
   - a. Recommended for publication 5
   - b. Withdrawn or dropped without prejudice 1
2. Manuscripts received during year 1958-59
   - a. Recommended for publication 29
   - b. In process of review 5
   - c. Withdrawn or dropped without prejudice 1

Other members of the Committee on Publications who handle the review of all papers submitted to the Division are as follows:

Adm. W. Mack Angas
Mr. Joseph M. Caldwell
Brig. Gen. Ellsworth I. Davis
Mr. James W. Dunham

DIVISION COMMITTEE ON RESEARCH SPONSORED FOUR PAPERS DURING YEAR

During the 1958-1959 year, the Division Committee on Research sponsored four papers at Society Conventions. Until January 1959, the committee was under the direction of Gen. Herbert D. Vogel, Chairman, Tennessee Valley Authority. Gen. Vogel was succeeded as Chairman by Col. Edmund H. Lang, who is assisted by Mr. Howard J. Marsden, Mr. Joe M. Caldwell, and Mr. Reed A. Elliot.

DIVISION MEMBERSHIP

In November 1955, 290 ASCE Members were registered in the Waterways and Harbors Division. In October 1959, Division membership had increased to 1,397, an increase of 381% in four years. The Division's excellent sessions at Conventions, outstanding journal contributions, and hard-working

committees have undoubtedly attracted many of these additional Division members.

COASTAL ENGINEERING REPORTS IN PROGRESS

Thorndike Saville, Jr., Chairman of the Division Committee on Coastal Engineering, and task committee chairmen Kenneth P. Peel (Task Committee on Groins) and Herbert C. Gee (Task Committee on Regulation of Coastal Structures) report that their attempts to put into print data on these subjects and on Sand By-Passing are continuing. Mr. Saville, Jr. is also chairman of the Task Committee on Sand By-Passing.

The Task Committee on Groins is continuing in its efforts to prepare a series of reports dealing with present practices in groin design and construction. The Task Committee on Sand By-Passing is continuing in its attempt to have published data on the various by-pass operations. A paper on the rather unique truck haul operation at Shark River, New Jersey, is scheduled for the New Orleans convention; another paper, on the Fire Island Inlet, New York, problem may be delivered at the 7th International Conference on Coastal Engineering (in the Netherlands) which, though not ASCE sponsored, is of interest to coastal engineers.

Review and editing of that section dealing with groins, of the Proceedings of the October 1958 seminar at Princeton, New Jersey, was completed early in the year. Review and editing of the rest of the Proceedings, dealing with sand by-passing, is still underway. Every effort is being made to complete this editing early next year.

Data on the state regulation of coastal structures have been received from all the governors of the 30 coastal states. These data are now being organized and summarized into comparable form, and will be sent back to the various states for their review and approval as to correctness and adequacy of the summarization.

COMMITTEE ON PORTS AND HARBORS AND ON NAVIGATION AND FLOOD CONTROL FACILITIES

The Division Committee on Ports and Harbors, under the direction of Ben E. Nutter, sponsored seven papers at two ASCE conventions during the 1958-1959 year. These papers covered a wide range of subjects. The committee also sponsored seven papers at the October Washington convention. Arrangements have been made for a symposium of wharf design for the New Orleans Convention (March 1960), and a symposium on marina development for the Reno Convention (June 1960).

The Committee on Navigation and Flood Control Facilities with Charles F. MacNish as Chairman sponsored eight papers covering subjects such as the Passamaquoddy Tidal Power project, navigation on the Great Lakes, flood control, and the like. Plans are well advanced for sessions at the New Orleans and Reno conventions.

COMMITTEE ON REGULATION AND STABILIZATION OF RIVERS EXPANDS ACTIVITIES

The Committee on Regulation and Stabilization of Rivers, following its reactivation in November 1958, sponsored two papers at Society Conventions. The Committee is also investigating current practices on stabilization of alluvial rivers and intends to encourage the presentation of papers on this subject. Mr. J. F. Friedkin is Chairman of this committee.

NEWSLETTER PUBLICATION

The next issue of the Waterways and Harbors Division Journal will be in March 1960. The deadline for submission of material for that issue is January 29, 1960. If you have any material which might be usable in the newsletter, please send it to:

Austin E. Brant, Jr.
Editor, Waterways and Harbors
Division Newsletter
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375 Park Avenue
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COMMITTEE ON REGULATION AND [illegible] OF FUTURES EXCHANGE ACTIVITIES

The Committee on Regulation and [illegible] of futures, following its [illegible]. The Committee is also investigating [illegible].

[illegible]

COMMITTEE ON PORTS AND HARBORS AND/OR [illegible]

The [illegible] Committee on Ports and Harbors, under the chairmanship of [illegible], [illegible] during the [illegible] year. [illegible]

The Committee on Navigation and Flood Control Facilities with [illegible], Chairman, [illegible] subjects such as the [illegible] project, navigation in the Great Lakes, [illegible], and the [illegible].